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Volume 5 | Technical Appendices

CFA20 | Curdworth to Middleton

Curdworth to Middleton river modelling report (WR-004-013)

Water resources

November 2013

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Appendix WR-004-013

Environmental topic:	Water resources and flood risk assessment	WR
Appendix name:	Modelling	004
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1 Overarching modelling approach

1.1 Introduction

1.1.1 This section of the Proposed Scheme crosses numerous watercourses with the potential for affecting flood risk. Hydraulic modelling has been carried out to assess the current (baseline) river flood risks at each of these watercourse crossings and the potential impacts of the proposed culvert and viaduct structures. Therefore, the primary objective of this assessment was to assess the impact of the Proposed Scheme on river flood risk.

1.1.2 The outcome of this assessment will aid the design to determine the type and dimension of structures required to convey the watercourse flows; and mitigation measures for any remaining residual flood risk.

1.1.3 A hydraulic modelling assessment of flood risk was undertaken for watercourses affected by this section of the Proposed Scheme. These watercourses were grouped into seven community forum areas (CFA) in this section of the Proposed Scheme. Existing hydraulic models of the watercourses have been utilised where available; and new river hydraulic models were built for the other watercourses. This report describes the hydraulic modelling processes and outcomes of this assessment.

1.1.4 The main conclusions from this modelling report form the basis of the river flood risk in the flood risk assessment (FRA) for the Curdworth to Middleton area (CFA20) (Volume 5: Appendix WR-003-020). These conclusions are also reported within the water resources and flood risk assessment section of Volume 2 of the Environmental Statement (ES).

1.2 Hydrology

1.2.1 Watercourses with existing hydraulic models adopted standard Flood Estimation Handbook (FEH) techniques for hydrological assessment. The hydrology of these models was reviewed for suitability for use in this study.

1.2.2 For the watercourses with no existing hydraulic models, hydrological assessments were undertaken in this assessment to determine the design flows.

1.2.3 The hydrological catchments of the watercourses to each of the route crossings have been determined from the FEH CD-ROM[1] for watercourses represented in this data set. For the purposes of this assessment it was assumed that catchment boundaries as represented in the FEH CD-ROM were correct, therefore a detailed assessment of catchment boundaries has not been completed. The catchment descriptors have also been taken from the FEH CD-ROM and updated for urban expansion to 2012, using Equation 6.8 in Volume 5 of the FEH[2]. This is a standard industry technique.

1.2.4 River flows at watercourse crossing locations were determined using the Revitalised Flood Hydrograph (ReFH) method[3] in the first instance. In line with the current Environment Agency flood estimation guidance, the ReFH method is deemed

[1] Centre for Ecology and Hydrology (2009), *FEH CD-ROM Version 3, ©NERC (CEH)*.
[2] Centre for Ecology & Hydrology (1999), *Flood Estimation Handbook – Volume 5: Catchment Descriptors*.
[3] Centre for Ecology & Hydrology (2007), *The revitalised FSR/FEH rainfall-run-off method: Supplementary Report No. 1*.

acceptable for the majority of catchments along the route and is the most time efficient method for determining flows for studies where numerous flows are required.

1.2.5 The ReFH method is not considered acceptable for all catchments, in this case those classed as highly permeable. Based on the FEH CD-ROM catchment descriptors, a number of the catchments are classed as highly permeable and hence in line with current Environment Agency guidelines (2012)[4], an alternative method was required. Therefore at these locations, the FEH Statistical method, with a permeable adjustment was utilised, as recommended in the guidelines.

1.2.6 Not all watercourses crossed by the route were represented in the FEH CD-ROM; therefore, the catchment boundaries could not be determined using the FEH CD-ROM. In these instances, catchment boundaries have been determined through the use of topographic data from Light Detection and Ranging (LiDAR) data and Ordnance Survey (OS) mapping at a 1:10,000 scale. At locations of uncertainty, a slightly larger catchment has been assumed as a conservative approach. Flows for these catchments were determined through a conservative area scaling method. Based on the flows estimated for FEH CD-ROM represented catchments, a maximum flow rate of 1.4 and 2.6m^3/s per km^2 was calculated for the 1 in 100 (1%) annual probability and 1 in 1000 (0.1%) annual probability events respectively. These flows rates, along with a 10% error allowance (to prevent an underestimation of flow), were used as scaling factors.

1.2.7 The estimated peak flows were used as either a constant inflow boundary or as a full hydrograph. The peak flows estimated using this method were for the 1 in 20 (5%) annual probability, 1 in 100 (1%) annual probability and 1 in 1000 (0.1%) annual probability events. Flow during the 1 in 100 (1%) annual probability event with an allowance for climate change was estimated by factoring the 1 in 100 (1%) annual probability flow by 20% (refer to the FRA in Volume 5: Appendix WR-003-020).

1.3 Hydraulics

General approach

1.3.1 The hydraulic modelling approach depended on the characteristics of the particular watercourse and floodplain hydraulics. The approach of either steady or unsteady modelling was based on whether there were rapid increases or decreases in flows, flood storage areas or structure impacts on channel/floodplain flows. The modelling approach also varied based on requirements of assessing the flow routes either in one dimension or two-dimensions.

1.3.2 The modelling approach adopted in this study was as follows:

- if the modelling was utilised for sizing the culvert crossings on watercourses with no significant floodplain attenuation or structure impacts, steady state one dimensional modelling was adopted;
- if there was significant floodplain attenuation and/or structure impacts on channel/floodplain flows, one dimensional hydrodynamic modelling was

[4] Environment Agency (2012), *Flood estimation guidelines* (197_08).

adopted; and

- if there was significant floodplain attenuation and/or structure impacts on channel/floodplain flows, and a requirement for accurately defining the flood extents, two dimensional or a one-dimensional – two dimensional combination modelling was adopted.

1.3.3 Existing models were first reviewed to assess their suitability for use. If more recent data such as topography was available the models were updated accordingly. If the level of detail within the model, such as the floodplain, was not appropriate, the model was upgraded accordingly.

1.3.4 The hydraulic modelling approach was based on the Environment Agency guidelines[5].

1.3.5 Two industry standard modelling packages have been utilised as part of this assessment: ISIS version 3.6 and TUFLOW 2012.

Hierarchical approach

1.3.6 Any existing Environment Agency models for the watercourses were used to assess the current and future flood risk impacts of the route crossing any watercourses.

1.3.7 For watercourses without existing hydraulic models, the modelling process was carried out in a phased manner to assess the baseline flood risk and impacts of the Proposed Scheme. In the first phase, the watercourses with culverted crossings were modelled as simple unsteady one dimensional hydraulic models, to assess the adequacy of culverts in conveying flood flows. In the second phase, watercourses for both culverted and viaduct crossings were modelled as two dimensional hydrodynamic models to define the flood extents and assess the impacts of the various structures on flood risk. The two dimensional model outputs were then used to inform the design team of flood risk.

1.3.8 All the models were run for the 1 in 100 (1%) annual probability with an allowance for climate change and 1 in 1000 (0.1%) annual probability events. Some of the models were run for the 1 in 20 (5%) annual probability event where the potential impacts on flood risk could affect vulnerable receptors.

1.3.9 The 1 in 100 (1%) annual probability with an allowance for climate change peak water levels for the baseline and Proposed Scheme were compared upstream and downstream of the crossing to assess the impact on flood risk. The Proposed Scheme impact on flood risk and the width of the 1 in 100 (1%) annual probability with an allowance for climate change flood extents, defined the type of structure to be used at the crossings i.e. culvert or viaduct and the dimensions of culverts/viaducts. The structure type was selected based on its adequacy in conveying flood flows without significantly affecting flood risk.

1.3.10 The peak water levels for the 1 in 1000 (0.1%) annual probability event confirmed whether the vertical alignment met the design criteria (refer to 'Section 3 of the Flood Risk Assessment in Volume 5: Appendix WR-003-020).

[5] Environment Agency (August 2009), *'Requirements for completing computer river modelling for flood risk assessments – Guidance for developers' Version 3.0.*

Input data

1.3.11 The topographic data used was LiDAR data that was flown in 2012, covering the extent of the Proposed Scheme, providing data as fine as up to 0.2m horizontal resolution. This data was used to create digital terrain models (DTM) for use within the hydraulic models. In most cases, the DTM has been resized to a 1m resolution for suitability in two dimensional models. For watercourses without existing hydraulic models, there were no topographic surveys available and hence river sections and floodplain topography were derived from this DTM.

1.3.12 For existing models, the floodplain topography was updated with this DTM. The channel topography in these models was taken from topographic surveys undertaken previously.

1.3.13 Inflows to the watercourses were taken from the hydrological assessments as discussed in Section 1.2 of this report.

1.3.14 The data for the Proposed Scheme model scenario was taken from the scheme drawings.

One dimension modelling

1.3.15 In the first phase, one dimensional ISIS models were constructed representing a 200m to 300m reach of the watercourse. The purpose of these models was to assess the adequacy of culverted crossings in conveying flows. These models used the LiDAR data to define extended cross-sections which included the channel and floodplain topography. The roughness of the channels and floodplains is defined by the Manning's roughness parameter. The Manning's values of channels and floodplains were based on the particular land use type as observed from aerial photographs. Steady state flows were applied as upstream inflow boundaries and a normal depth boundary was applied at the downstream extent. The normal depth boundary was based on the bed slope of the topography at that location and is considered suitable for the purpose of the modelling.

1.3.16 The Proposed Scheme model included rectangular conduit units to represent the structures at the crossings. There were two types of culverts adopted: a minimum culvert size of 2m by 1.5m and a maximum culvert size of 4m by 2m. The dimensions adopted here represent the flow area of the culvert rather than the full dimensions of the culvert that would need to be larger to accommodate depressed inverts and mammal ledges as appropriate. The lengths of the culvert were based on the width of the route crossings as defined in the Proposed Scheme design.

Two dimension modelling

1.3.17 In the second phase, unsteady state two dimensional TUFLOW models were built to accurately define the flood extents and floodplain attenuation. The two dimensional models were built on a 5m cell resolution with LiDAR data used to create the DTM, which defined the floodplain and channel topography.

1.3.18 It should be noted that components within a two dimensional TUFLOW model such as SXZ, HX, Z-polygon, Z-Shape polygons, etc., are based on naming conventions as defined in the TUFLOW manual[6].

1.3.19 The Manning's roughness values of the channels and floodplains were based on the particular land use type as observed from aerial photographs.

1.3.20 The inflow to each watercourse was applied upstream using a TUFLOW boundary condition polyline layer, linking it to a flow time series within a boundary condition database. The flow type is either constant flow or hydrograph flow, depending on the attenuation within the floodplain. A flow-head (HQ) polyline layer was used for the downstream boundary, based on the slope of the floodplain at that location; which was considered suitable for the scale and level of detail of the modelling. The models have been run at a two second timestep for varying durations.

1.3.21 The Proposed Scheme model was built by adding either culvert or viaduct structures to the baseline model at the watercourse crossings.

1.3.22 Viaduct structures have been modelled by adding route embankments as Z-polygon or Z-Shape polygon layers with an opening at the viaduct crossing. The Z-polygon or Z-Shape polygon layers are Geographic Information System (GIS) polygons with elevations. Where piers were modelled, they were represented as Flow Constriction (FC) shape layers. The soffit levels were not added into the model. This was because the 1 in 1000 (0.1%) annual probability modelled peak flood levels, along with sufficient clearance, will form the basis of designing the soffit heights (refer to Section 3 ' of the Flood Risk Assessment in Volume 5: Appendix WR-003-020).

1.3.23 Culvert structures have been modelled by adding a one dimensional network layer representing the extent of the culvert, the length of which was determined by the width of the route at the crossing point (including embankment earthworks and any landscaping). Inverts were defined at the inflow and outflow points of the culvert extracted from the LiDAR DTM for the area. This one dimensional network layer was connected to the two dimensional domain with a 'SXZ' point link, a GIS point used in the modelling software for one dimensional-two dimensional linking. An embankment was modelled across the route as a Z-polygon layer, covering the extent of the upstream floodplain at the route crossing so that all flow was routed through the culvert.

One dimension-two dimension linked modelling

1.3.24 In certain cases where existing one dimensional models were not representing complex channel-floodplain interactions accurately, dynamically linked one dimensional two dimensional models were constructed. The channel component was represented in one dimension and the floodplain component in two dimensional. These models were built using ISIS-TUFLOW.

1.3.25 The flows between the one dimensional and two dimensional model components were controlled via a GIS polyline layer ('HX' layer), the spill levels of which are defined by the channel bank levels or DTM levels.

[6] BMT WBM (2010), *TUFLOW User Manual, 2010*.

1.3.26 In the Proposed Scheme scenarios, the viaduct structures are represented as discussed earlier in the two dimensional modelling section (Section 1.3.22 of this report).

Sensitivity assessments

1.3.27 Sensitivity assessments have been undertaken on various parameters of the models to reflect the uncertainties and impacts on modelled flood levels. Assessments have been carried out on inflows and culvert blockages. In the case of viaduct crossings, sensitivity was undertaken on inflows.

1.3.28 Sensitivity assessment on inflows was carried out by varying the 1 in 100 (1%) annual probability with an allowance for climate change and the 1 in 1000 (0.1%) annual probability flows by 20%. This was undertaken for the baseline and Proposed Scheme scenarios, unless stated otherwise.

1.3.29 Sensitivity assessment has also been carried out on Proposed Scheme scenarios with culvert structures by adding 10% blockage. Resulting models have been run for the 1 in 100 (1%) annual probability with an allowance for climate change and the 1 in 1000 (0.1%) annual probability events.

1.4 Assumptions and limitations

Hydrology

1.4.1 The catchment boundaries as taken from the FEH CD-ROM are correct and accurately represent the catchments in reality.

1.4.2 For catchments not classed as highly permeable, the ReFH method results in the most accurate estimation of flow at the location of the crossings in comparison to other methods.

1.4.3 The FEH Statistical method with permeable adjustment results in the most accurate estimation of flow at catchments classed as highly permeable.

1.4.4 The area scaling method, which is based on area, results in conservative flow estimates for catchments which are not represented in the FEH CD-ROM (refer to Section 1.2 of this report for detail).

1.4.5 There are no external influences on flow at the location of the crossing, such as significant abstractions or discharges.

1.4.6 A 20% allowance for climate change on peak flow rates has been adopted for the 1 in 100 (1%) annual probability with an allowance for climate change event.

Hydraulic modelling

1.4.7 Only river flood risk was considered during the hydraulic modelling in this assessment.

1.4.8 For watercourses without existing hydraulic models, the watercourse geometry was extracted from the LiDAR DTM with the channel width defined by the 5m cell resolution of the two dimensional model. Therefore, the watercourse geometry is not well defined, the consequence of which is an underestimate of the channel conveyance and hence, an overestimation of the floodplain inundation.

1.4.9 There will be certain watercourses with road crossing structures upstream or downstream of a route crossing, which will cause a significant impact on hydraulics. OS mapping and aerial photography were used to assess the location of the structures. The inverts of any culvert structure were assumed to be the channel bed levels from the LiDAR DTM; and structure widths as the width of the channel.

1.4.10 In the Proposed Scheme for models involving viaducts, the structure was represented by the piers and embankments. The scheme drawings were used to obtain the footprint of the piers and the dimensions incorporated into the model. The soffits of the viaducts were not modelled as the design approach for the structures is to include a suitable clearance between peak flood level and the structure soffit.

2 Modelling at watercourse crossings

2.1 Overview

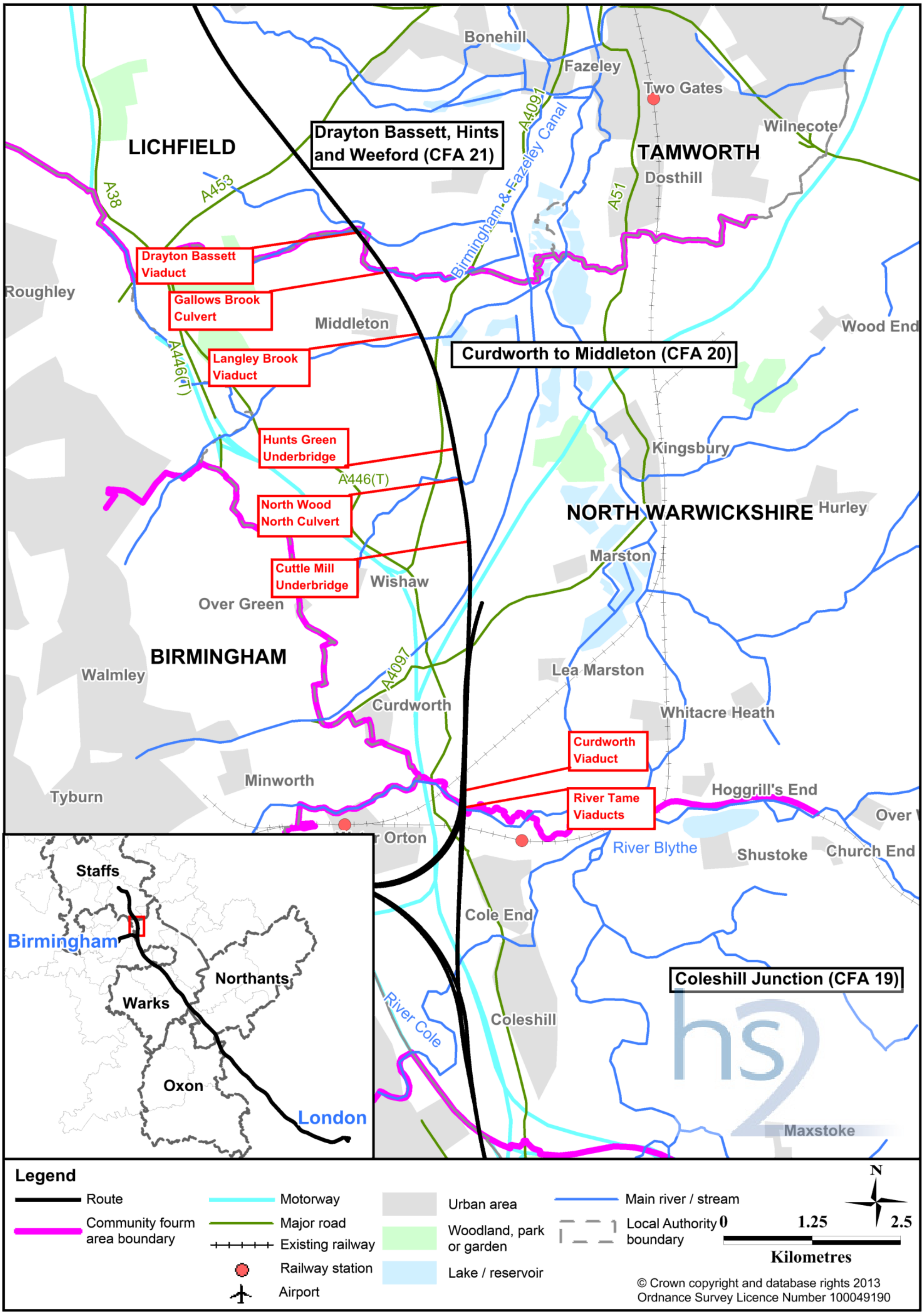
2.1.1 River modelling undertaken at the various watercourse crossings for the Curdworth to Middleton area are summarised in Table 1, along with the modelling methodologies adopted. The River Tame viaducts mentioned in Table 1 comprise of the River Tame west viaduct, River Tame east viaduct and Water Orton no.2 viaduct. Figure 1 identifies the location of each of these structures.

Table 1: River models at watercourse crossings

Crossing name	Watercourse identifier and map reference	Watercourse	Hydrology	Hydraulic modelling
River Tame viaducts	SWC-CFA20-001 (Volume 5: Map WR-05-056, H6)	Main river (River Tame)	FEH statistical	one dimensional hydrodynamic
Curdworth viaduct	SWC-CFA20-002 (Volume 5: Map WR-05-056, G6)	Ordinary watercourse (tributary of the River Tame)	ReFH	one dimensional hydrodynamic
Curdworth viaduct	SWC-CFA20-003 (Volume 5: Map WR-05-056, G6)	Ordinary watercourse (tributary of the River Tame)	ReFH	one dimensional hydrodynamic
Cuttle Mill underbridge	SWC-CFA20-009 (Volume 5: Map WR-05-057, F5)	Ordinary watercourse (tributary of Langley Brook)	ReFH	two dimensional hydrodynamic
North Wood north culvert	SWC-CFA20-010 (Volume 5: Map WR-05-057, D6)	Ordinary watercourse (tributary of Langley Brook)	ReFH	two dimensional hydrodynamic
Hunts Green underbridge	SWC-CFA20-011 (Volume 5: Map WR-05-057, C6)	Ordinary watercourse (tributary of Langley Brook)	ReFH	two dimensional hydrodynamic
Langley Brook viaduct	SWC-CFA20-013 (Volume 5: Map WR-05-058, F6)	Ordinary watercourse (Langley Brook)	ReFH	two dimensional hydrodynamic
Gallows Brook culvert	SWC-CFA20-014 (Volume 5: Map WR-05-058, C6)	Ordinary watercourse (Gallows Brook)	ReFH	two dimensional hydrodynamic
Drayton Bassett viaduct	SWC-CFA20-015 (Volume 5: Map WR-05-058, B7)	Ordinary watercourse (tributary of the River Tame)	ReFH	two dimensional hydrodynamic

2.1.2 A summary of the modelling for the Gallows Brook culvert is provided in Section 2.2 of this report. The modelling is described in detail for each of the viaduct structures from Sections 2.3 to 2.8 of this report. This includes details of the specific modelling methodologies, hydraulic constraints and any assumptions on each of the watercourse crossings.

Figure 1: Location plan



2.2 Culverts

2.2.1 A two dimensional TUFLOW model was built for the watercourse at Gallows Brook culvert. The two dimensional TUFLOW hydraulic models built for the baseline and Proposed Scheme scenarios used the general methodologies for two dimensional modelling as discussed in Section 1.3 in this report.

2.2.2 The details of the hydrological assessment at this crossing are provided in the FEH proforma in Section 0 of this report.

2.2.3 The structure adopted at the culvert crossing along with the impacts on peak flood levels are summarised in Table 2. The structure dimensions of width (W), height (H) and length (L) in metres is also provided in this table.

Table 2: Modelled peak levels at culvert crossings

Structure	Watercourse identifier	Structure dimensions (WxHxL)	Flood event	Peak flood level (mAOD)		Change in flood level (mm)	Length of impact upstream reach[7] (m)
				Baseline	Scheme		
Gallows Brook culvert	SWC-CFA20-014	2m x 1.5m x43m	1 in 20 (5%)	78.791	78.791	0	No impact
			1 in 100 (1%) climate change	78.796	78.796	0	
			1 in 1000 (0.1%)	78.819	78.807	-12	

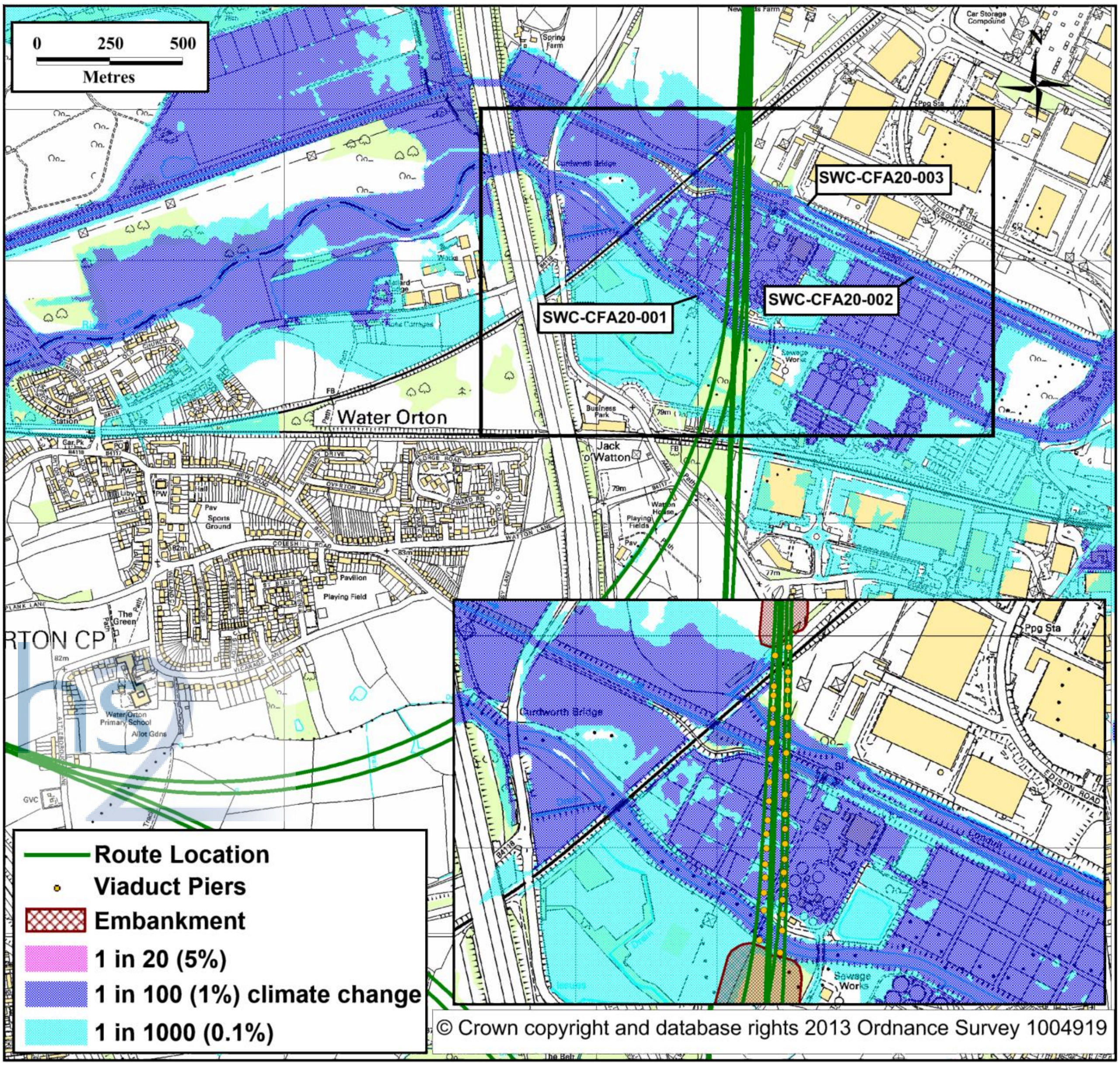
2.2.4 For Gallows Brook culvert there is no increase in peak levels for the 1 in 100 (1%) annual probability event. Therefore, the culvert structure does not increase flood risk and has adequate capacity to convey flood flows.

2.3 River Tame viaducts and Curdworth viaduct

2.3.1 This crossing consists of the River Tame viaduct structures (River Tame west viaduct, River Tame east viaduct and Water Orton no.2 viaduct) over the River Tame, SWC-CFA20-001 (Volume 5: Map WR-05-056, H6). It also consists of the Curdworth viaduct structure crossing watercourses SWC-CFA20-002 (Volume 5: Map WR-05-056, G6) and SWC-CFA20-003 (Volume 5: Map WR-05-056, G6). These watercourses flow from west of the crossing and continue east as shown in Figure 2.

[7] Length of reach upstream of the scheme along which flood levels during the 1 in 100 (1%) annual probability with an allowance for climate change event are greater than 10mm.

Figure 2: Crossing location plan for River Tame viaducts and Curdworth viaduct



Hydrology

2.3.2 The hydrology of the River Tame was taken from the Environment Agency Flood Risk Mapping Study[8]. The hydrological assessment undertaken as a part of the flood risk mapping study and used standard FEH methods: the Statistical method and the Flood Studies Report Rainfall Run-off method. The Statistical method was used to estimate the peak design flows and the Rainfall Run-off method to generate hydrographs scaled to these peak flow estimates. For the final flood frequency curves, single site analysis was adopted for lower design events, and pooled analysis for higher design events. The hydrology was calibrated by comparing the modelled and observed ratings at various gauges.

2.3.3 There are no known hydrological studies for the two tributaries at Curdworth viaduct. Therefore the use of previously estimated flows was not possible. Similarly no suitable anecdotal records were available to inform the hydrological study. The hydrological

assessment adopted is discussed in detail in Section 0 of this report. The flows at this crossing are provided in Table 3.

Table 3: Modelled inflows for River Tame viaducts and Curdworth viaduct

Watercourse identifier	Environment Agency Flood Zone	1 in 100 (1%) climate change flow	1 in 1000 (0.1%) flow	Modelled structure
SWC-CFA20-001	3	210.74m^3/s	312.67m^3/s	Viaduct
SWC-CFA20-002	3	5.42m^3/s	8.23m^3/s	Viaduct
SWC-CFA20-003	3	0.38m^3/s	0.73m^3/s	Viaduct

Hydraulics

2.3.4 The hydraulic model available at the time of this assessment was from the Environment Agency Flood Risk Mapping Study[8] undertaken by Halcrow in 2009. There were two hydraulic models covering the Upper Tame and Lower Tame which were built using ISIS version 3.0. There was an additional two dimensional model using ISIS-TUFLOW at Bescot Junction and Brookvale Road, Witton, which gave a better understanding of the complex flow regime at this location. However, the two dimensional model extent does not cover the reach of the River Tame near the crossing. Therefore, the Lower Tame one dimensional ISIS model was used for the purposes of this assessment as this covered the study area crossing. The one dimensional model was adequate in defining the floodplain hydraulics at this location. The two tributaries that have their confluence with the River Tame at Curdworth viaduct were not modelled explicitly but were included as inflow points to the Lower Tame model as a part of this assessment.

2.3.5 The Lower Tame ISIS model was amended at the proposed crossing to improve the floodplain hydraulics and includes the two tributaries at Curdworth viaduct as point inflows. The extended cross-sections near the proposed crossing were replaced by channel sections with ISIS reservoir units representing the floodplain areas. There were two floodplain areas located upstream and downstream of the crossing and linked together by an ISIS floodplain section. The level-area relationship of the floodplain areas and topography of the floodplain section were extracted from the most recent DTM available for this study (refer to Section 1.3.11 of this report).

2.3.6 The proposed viaduct structure was modelled by modifying the floodplain section data to include pier widths and embankment sections. The soffit levels were not added into the model. This was because the 1 in 1000 (0.1%) annual probability modelled peak flood levels, along with sufficient clearance, would form the basis of designing the soffit heights.

2.3.7 The main hydraulic constraint is the railway bridge 300m upstream of the viaduct crossing which has been represented in the Environment Agency ISIS model, utilising survey data.

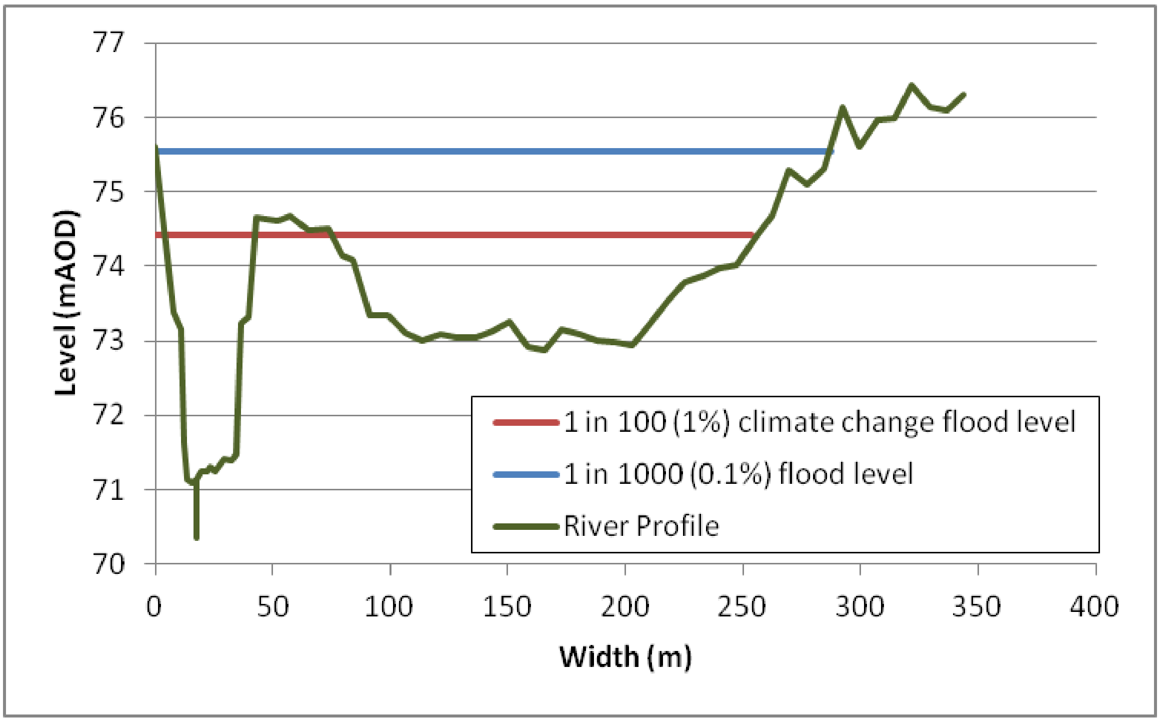
2.3.8 The baseline floodplain width at the crossing is 494m for the 1 in 100 (1%) annual probability with an allowance for climate change event. The cross-section at each of

[8] Environment Agency (2009), *River Tame Hazard Mapping*. Completed by Halcrow on behalf of the Environment Agency.

the viaduct structures and peak flood levels are provided in Figure 3. The modelled peak levels and impacts of the Proposed Scheme are summarised in Table 4.

Figure 3: Cross-section and flood levels at River Tame viaducts and Curdworth viaduct

Cross-section at SWC-CFA20-001



Cross-section at SWC-CFA20-002 and SWC-CFA20-003

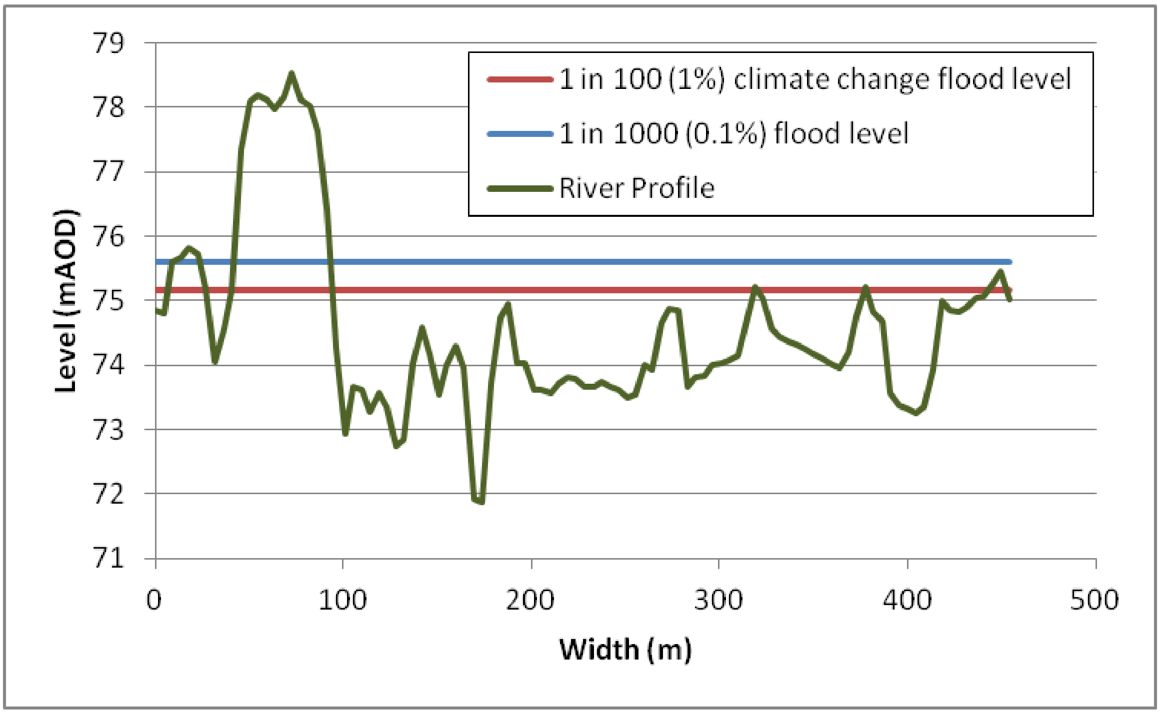


Table 4: Modelled peak levels for River Tame viaducts and Curdworth viaduct

Flood event	Watercourse identifier	Peak flood level		Change in flood level
		Baseline	Scheme	
1 in 100 (1%) climate change	SWC-CFA20-001	74.415mAOD	74.415mAOD	0mm
	SWC-CFA20-002	75.153mAOD	75.153mAOD	0mm
	SWC-CFA20-003	75.153mAOD	75.153mAOD	0mm
1 in 1000 (0.1%)	SWC-CFA20-001	75.543mAOD	75.543mAOD	0mm
	SWC-CFA20-002	75.598mAOD	75.600mAOD	2mm
	SWC-CFA20-003	75.598mAOD	75.600mAOD	2mm

Sensitivity assessment

2.3.9 Sensitivity assessment was carried out on inflows for the 1 in 100 (1%) annual probability as a part of the Environment Agency Flood Risk Mapping Study[8]. A 20% increase in inflows caused up to a 272mm increase in peak levels. However, the soffit level will be sufficiently above the modelled peak levels with sensitivity allowance, providing the design clearance of 600mm. Therefore, the sensitivity changes on peak levels will not affect the vertical alignment of the route.

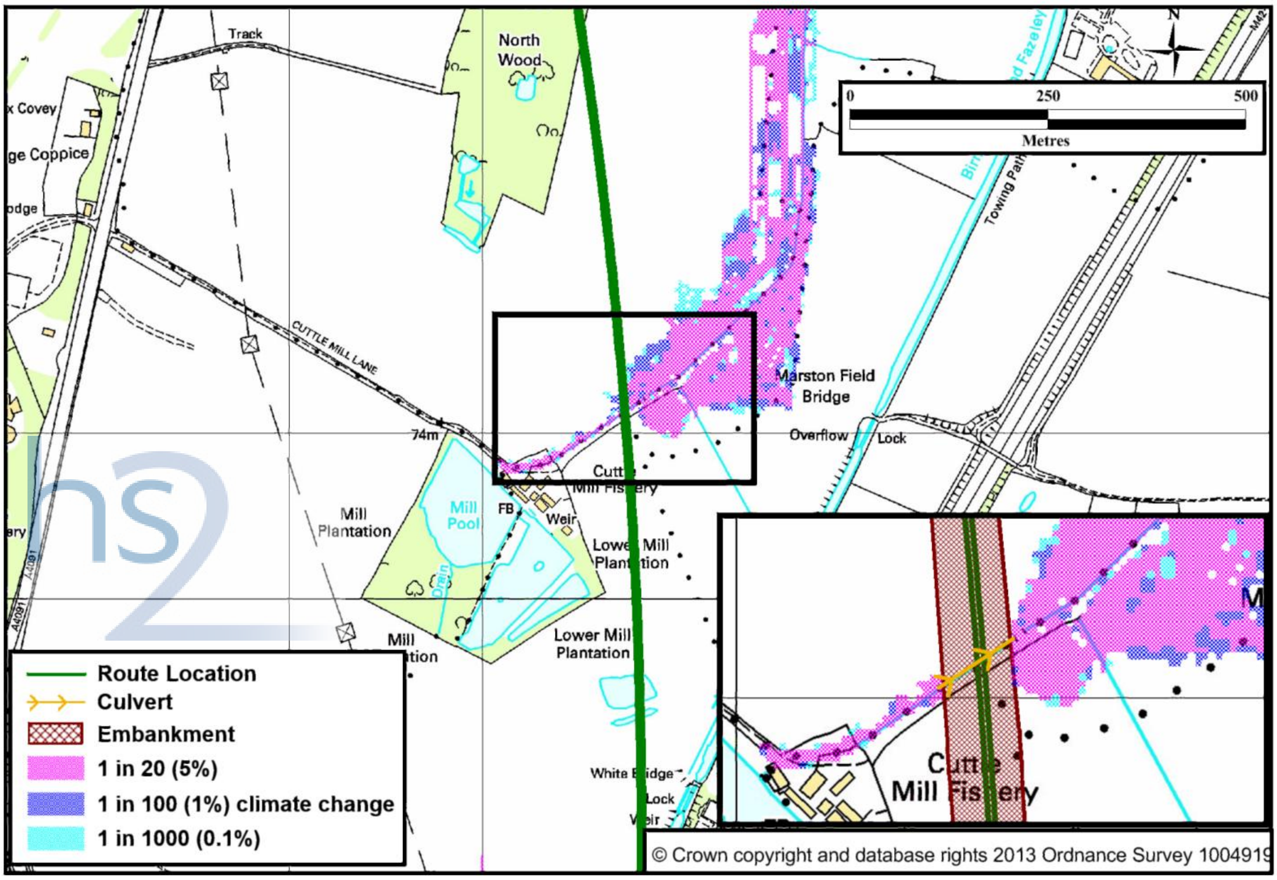
Conclusions

2.3.10 The Proposed Scheme showed no increase in peak levels for the 1 in 100 (1%) annual probability with an allowance for climate change event. Therefore, the viaduct structure at this location has no impact on flood risk.

2.4 Cuttle Mill underbridge

2.4.1 The crossing will consist of an underbridge of 25.44m width at watercourse SWC-CFA20-009 (Volume 5: Map WR-05-057, F5). The watercourse flows from west of the crossing and continues north-east as shown in Figure 4.

Figure 4: Crossing location and flood extents of Cuttle Mill underbridge



Hydrology

2.4.2 The river inflow hydrology was defined using the ReFH method. The hydrological assessment adopted the general methodology as described in Section 1.2.3 and Section 1.2.4 of this report. Details of the ReFH methodology are provided in Section 0 of this report. The peak flow from the hydrology calculation was used as a constant inflow into the model. The flows for this model are summarised in Table 5.

Table 5: Hydrology results: model inflows for Cuttle Mill underbridge

Watercourse identifier	Environment Agency Flood Zone	1 in 100 (1%) climate change flow	1 in 1000 (0.1%) flow	Modelled structure
SWC-CFA20-009	3	1.5m^3/s	2.37m^3/s	Underbridge

Hydraulics

2.4.3 A TUFLOW model was built on a 5m grid resolution. The lateral extents of the model were defined by the coverage of available 1m resolution LiDAR data. The inflows to these watercourses were applied upstream using a boundary condition polyline layer, linking it to a steady state flow time series within a boundary condition database. The downstream boundary was a HQ polyline layer based on the slope of the floodplain at that location; which was 1 in 200 in this case. The resulting baseline model for this particular watercourse was run at a two second timestep for the duration of five hours.

2.4.4 The Proposed Scheme at this crossing will be a 25.44m wide underbridge structure. However for the hydraulic assessment a 4m x 2m culvert was modelled. The culvert

was modelled as a one dimensional network layer within the two dimensional model, which used the LiDAR DTM to extract upstream and downstream inverts. The embankment at the crossing took into account of the full width of the route. The length of the culvert was set to the route width at the crossing.

2.4.5 The hydraulic constraints of this model are as follows:

- it has been assumed that the upstream lake at Mill Pool is full at the time of the flood events and so any attenuation from the upstream lake is not included; and
- apart from the upstream lake, there are no other physical constraints immediately upstream or downstream of the route crossing.

2.4.6 The baseline floodplain width at the crossing is 40m for a 1 in 100 (1%) annual probability with an allowance for climate change event. The cross-section with peak flood levels is shown in Figure 5. The modelled peak levels along with Proposed Scheme impacts are summarised in Table 6.

2.4.7 The baseline peak velocities and scheme impacts for the 1 in 100 (1%) annual probability with an allowance for climate change event are also provided in Figure 6.

Figure 5: Cross-section with flood levels for Cuttle Mill underbridge

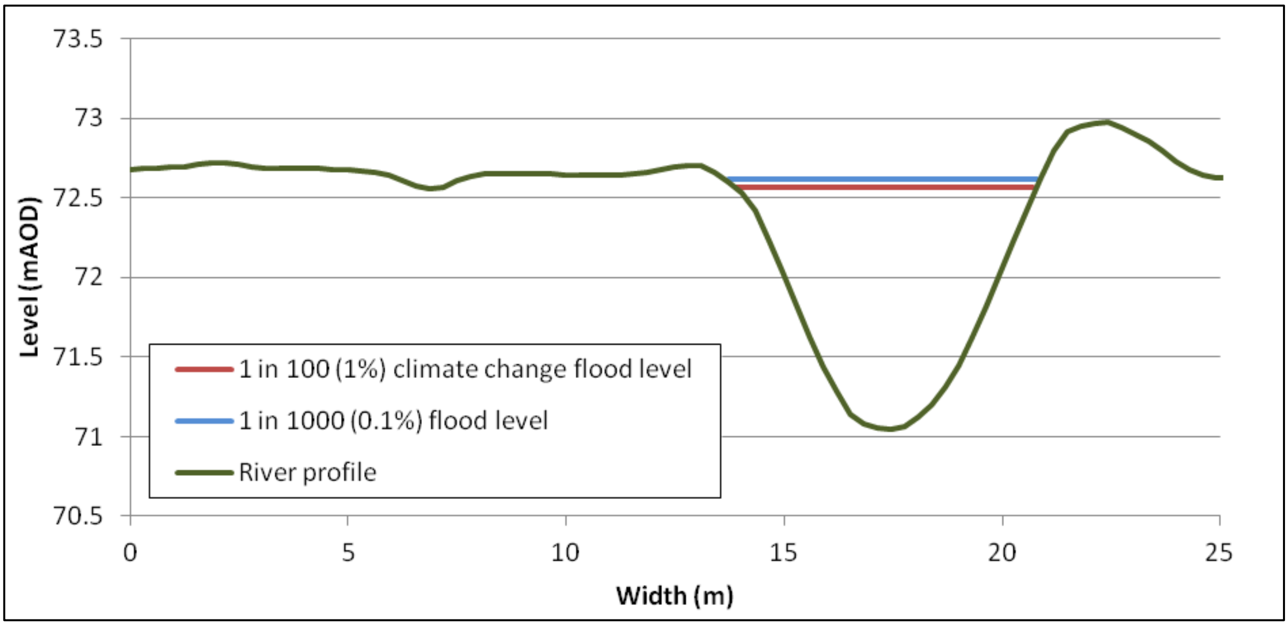
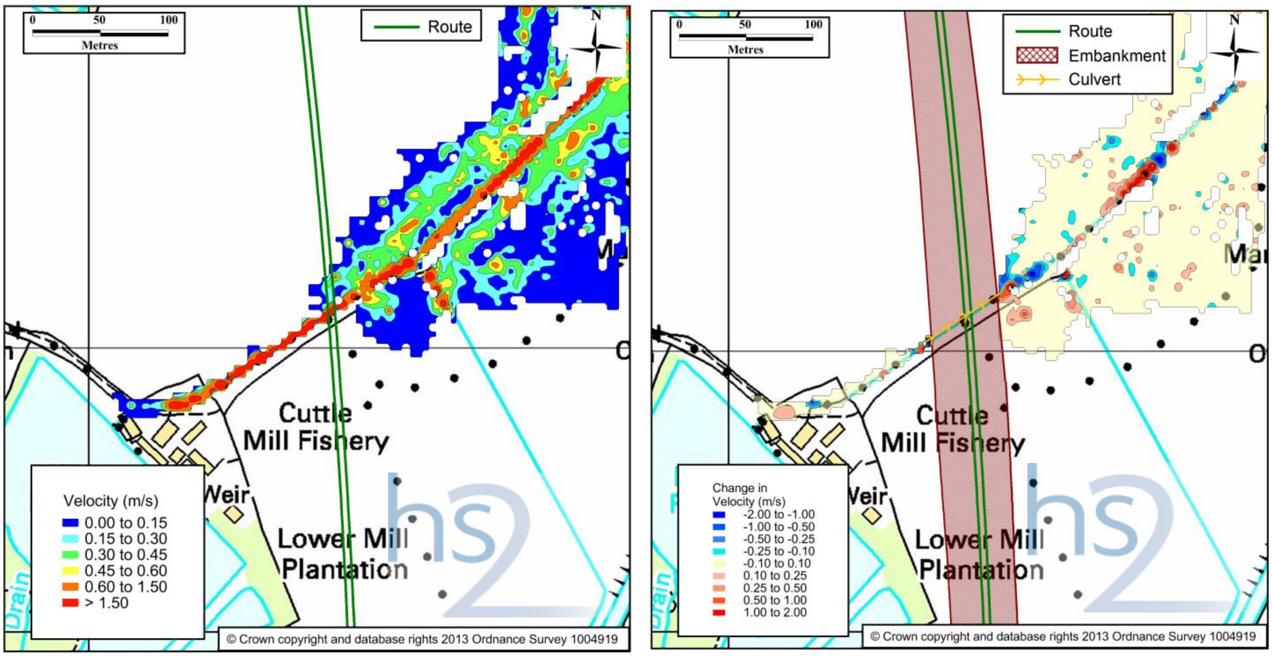


Table 6: Modelled peak levels for Cuttle Mill underbridge

Flood event	Peak flood level		Change in flood level
	Baseline	Scheme	
1 in 20 (5%)	72.504mAOD	72.503mAOD	-1mm
1 in 100 (1%) climate change	72.558mAOD	72.547mAOD	-11mm
1 in 1000 (0.1%)	72.616mAOD	72.670mAOD	54mm

Figure 6: Baseline peak velocity contours and scheme impact on velocities for 1 in 100 (%) climate change event at Cuttle Mill underbridge



Sensitivity assessment

2.4.8 Sensitivity was carried out by adding 20% to the 1 in 100 (1%) annual probability with an allowance for climate change and 1 in 1000 (0.1%) annual probability events. Models were run for both the baseline and scheme scenarios. Sensitivity on flows showed up to 28mm increase in peak level for 1 in 100 (1%) annual probability with an allowance for climate change. This change is minimal and hence the impact of the scheme on flood risk will still be valid with these sensitivity changes.

Conclusions

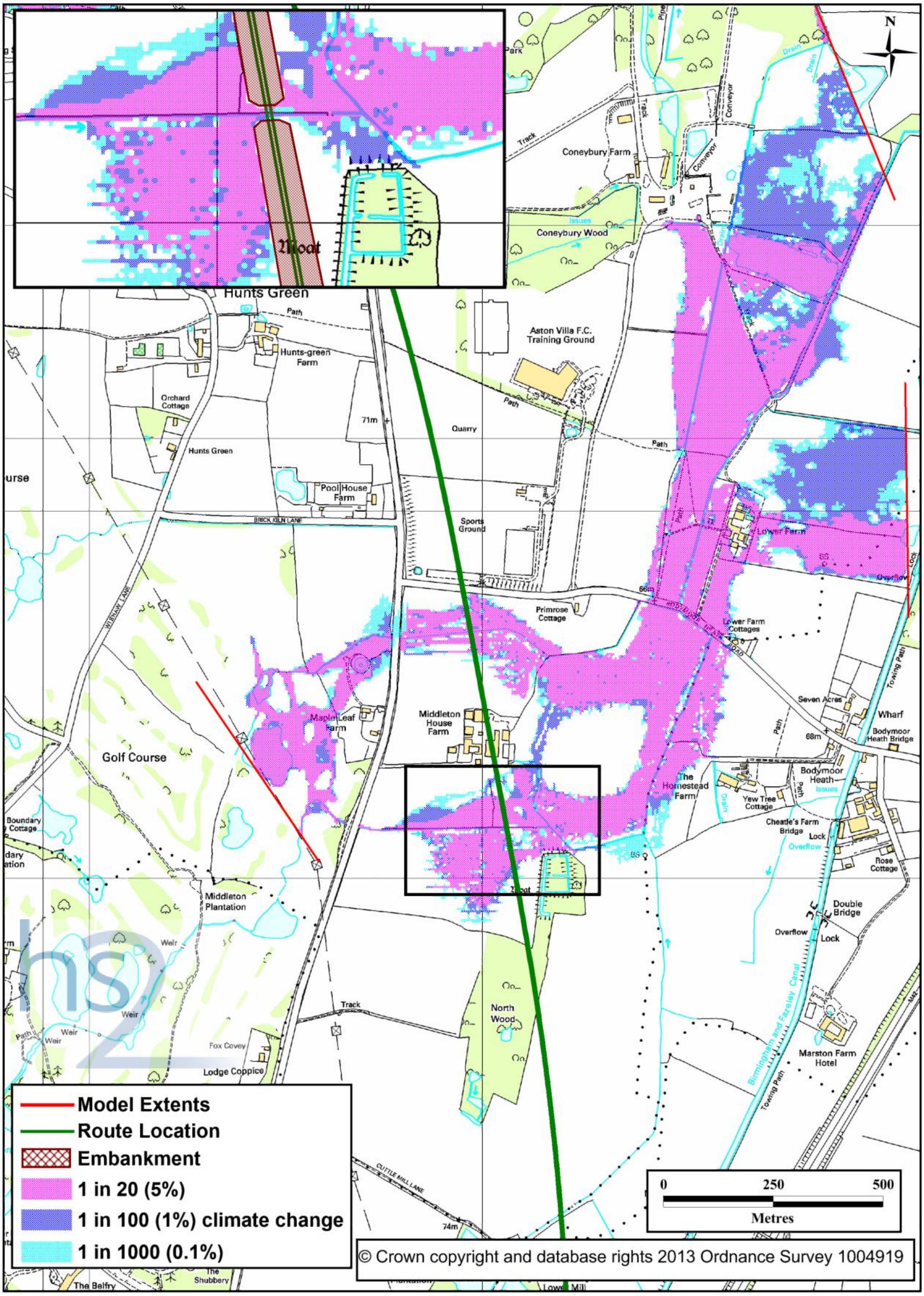
2.4.9 The Proposed Scheme shows a minimal increase in peak levels for the 1 in 100 (1%) annual probability event. Therefore, the viaduct structure has no impact on flood risk at this location.

2.4.10 There are localised increases of velocities of up 1.30m/s at the structure and minimal changes elsewhere.

2.5 North Wood north culvert

2.5.1 The crossing consists of a culvert of 13m width at watercourse SWC-CFA20-010 (Volume 5: Map WR-05-057, D6).The watercourse flows from west of the crossing and continues north-east within the model extents shown in Figure 7.

Figure 7: Crossing location and flood extents at North Wood north culvert



Hydrology

2.5.2 The river inflow hydrology was defined using the ReFH method. The hydrological assessment adopted the general methodology as described in Section 1.2.3 and Section 1.2.4 of this report. Details of the ReFH methodology are provided in Section 0 of this report. The estimated peak flows were set as hydrographs as an inflow boundary to the hydraulic model. The flow values are provided in Table 7.

Table 7: Hydrology results: modelled inflows for North Wood north culvert

Watercourse identifier	Environment Agency Flood Zone	1 in 20 (5%) flow	1 in 100 (1%) climate change flow	1 in 1000 (0.1%) flow	Modelled structure
SWC-CFA20-010	3	2.87m^3/s	4.96m^3/s	7.54m^3/s	Culvert

Hydraulics

2.5.3 The Environment Agency confirmed that there were no existing hydraulic models available for this particular watercourse. A TUFLOW model was built on a 5m grid resolution. The two dimensional domain covered the floodplain of the two watercourses at the crossings of North Wood north culvert and Hunts Green underbridge. The reason for their inclusion was the hydraulic interaction of these watercourses downstream of the crossing. The lateral extents of the model were defined by the coverage of available 1m resolution LiDAR data. A Manning's roughness coefficient of 0.04 was adopted for the floodplain and the watercourse. An HQ polyline layer has been used for the downstream boundary, based on the slope of the floodplain at that location; which was a 1 in 200 slope in this case.

2.5.4 The watercourse was not explicitly represented within the model but instead the LiDAR DTM was used to provide the channel geometry. The watercourse depth was defined by the LiDAR DTM and the width limited to the cell size width of 5m. Therefore, the watercourse geometry is not well defined, the consequence of which is an underestimate of the channel conveyance and hence, overestimate the floodplain inundation.

2.5.5 The watercourse reach upstream of the A4091 Tamworth Road Bridge was refined further to improve the flow route. This was achieved by using a GIS polyline layer or Z-line layer to smoothen the channel bed topography. In addition, the attenuation upstream of the A4091 Road Bridge was modelled by a Z-line layer representing the road culvert.

2.5.6 In the Proposed Scheme model, the route embankment was represented as a Z-shape layer with an arbitrarily high elevation. The culvert structure was represented as 13m wide in the route embankment. The soffit level was assumed to be sufficiently higher than the peak levels of the 1 in 1000 (0.1%) annual probability event.

2.5.7 The model was run at a two second timestep for duration of 20 hours.

2.5.8 It has been assumed that the upstream lake at Mill Pool is full at the time of the flood events and so any attenuation from the upstream lake is not included. Apart from the upstream lake, there are no other physical constraints immediately upstream or downstream of the route crossing.

2.5.9 The baseline floodplain width is 236m for the 1 in 100 (1%) annual probability with an allowance for climate change. The cross-section with flood levels at the crossing is provided in Figure 8. The modelled peak levels and scheme impacts are summarised in Table 8. The baseline peak velocity contours and scheme impacts on velocities for the 1 in 100 (1%) annual probability with an allowance for climate change event are also provided in Figure 9.

Figure 8: Cross-section with flood levels at North Wood north culvert

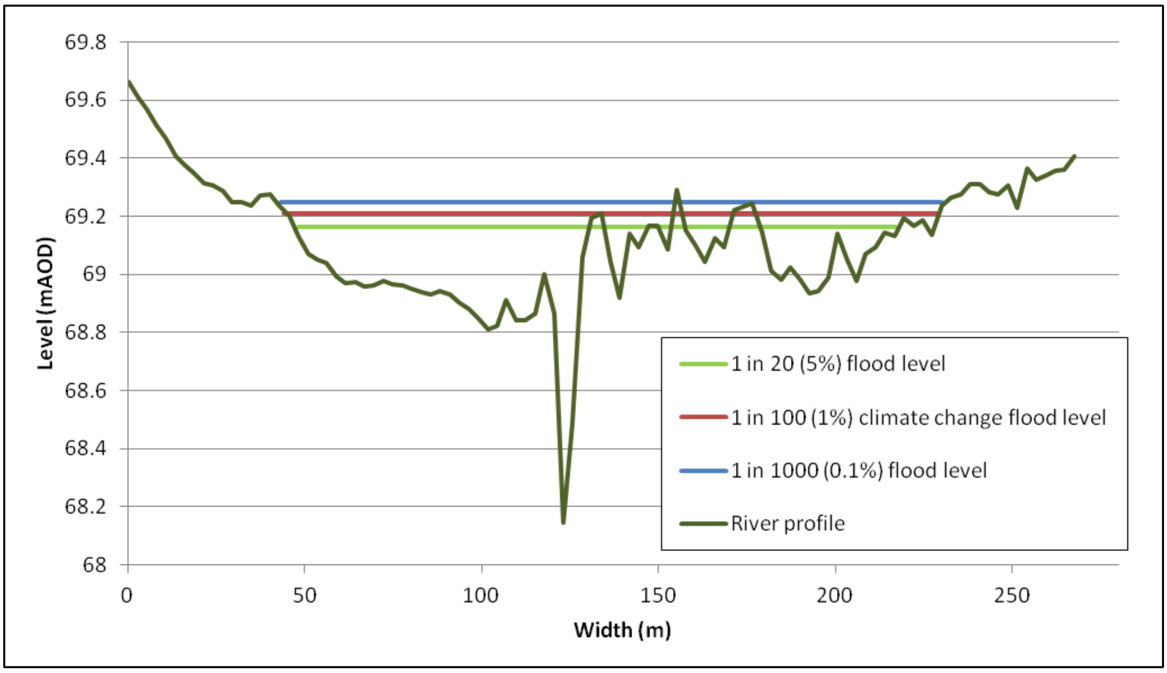
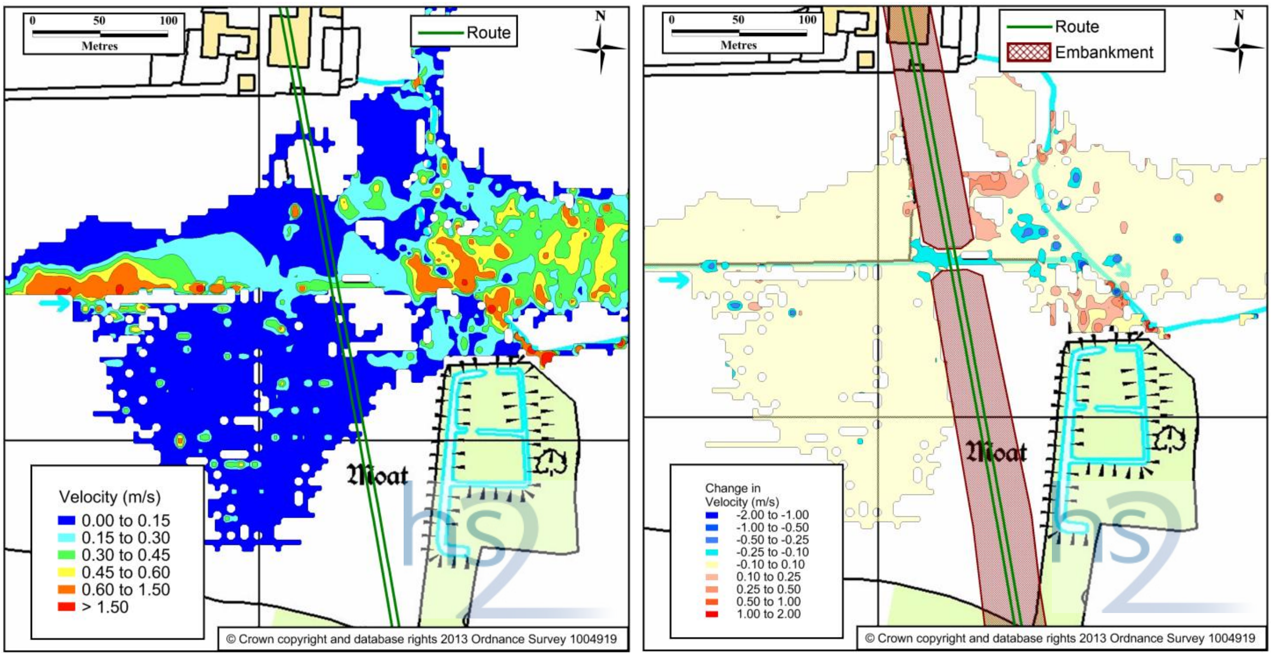


Table 8: Modelled peak levels for North Wood north culvert

Flood event	Peak flood level		Change in flood level
	Baseline	Scheme	
1 in 20 (5%)	69.162mAOD	69.173mAOD	11mm
1 in 100 (1%) climate change	69.211mAOD	69.252mAOD	41mm
1 in 1000 (0.1%)	69.249mAOD	69.347mAOD	98mm

Figure 9: Baseline peak velocity contours and scheme impact for 1 in 100 (1%) climate change event at North Wood north culvert



Sensitivity assessment

2.5.10 Sensitivity assessment was carried out by adding 20% to the 1 in 100 (1%) annual probability with an allowance for climate change and 1 in 1000 (0.1%) annual probability events. Models were run for both the baseline and Proposed Scheme scenarios. The increase in flows showed up to a 41mm rise in scheme peak levels for the 1 an 100 (1%) annual probability plus climate change event. However, the soffit level will be sufficiently above the modelled peak levels with sensitivity allowance, providing the design clearance of 600mm. There is a change in flood extents of up to 8% with no additional receptors being affected apart from agricultural land. Therefore, the impacts of the scheme on flood risk will still be valid with these sensitivity changes.

Conclusions

2.5.11 The Proposed Scheme shows an increase in peak levels of 41mm for the 1 in 100 (1%) annual plus climate change probability event. The increase in peak levels greater than 10mm is limited to a reach 140m in length upstream of the crossing.

2.5.12 An area has been identified which provides the adequate volume to replace the floodplain volume lost due to the scheme. With mitigation, the scheme shows an 18mm decrease in 1 in 100 (1%) annual probability with an allowance for climate change event.

2.5.13 There is a localised increase in peak velocities just downstream of the crossing of up to 0.3m/s for the 1 in 100 (1%) annual probability with an allowance for climate change event. There were minimal changes elsewhere.

2.6 Hunts Green underbridge

2.6.1 The crossing consists of a 4m x 3.35m culvert structure at watercourse SWC-CFA20-011 (Volume 5: Map WR-05-057, C6). The watercourse flows from west of the crossing and continues east within the modelled extent shown in Figure 10.

Hydrology

2.6.2 The river inflow hydrology was defined using the ReFH method. The hydrological assessment adopted the general methodology as described in Section 1.2.3 and Section 1.2.4 of this report. Details of the ReFH methodology are provided in Section 0 of this report. The estimated peak flows were set as hydrographs as an inflow boundary to the hydraulic model.

Table 9: Hydrology results: Modelled inflows for Hunts Green underbridge

Watercourse identifier	Environment Agency Flood Zone	1 in 20 (5%) flow	1 in 100 (1%) climate change flow	1 in 1000 (0.1%) flow	Modelled structure
SWC-CFA20-011	3	1.51m^3/s	2.64m^3/s	4.09m^3/s	Underbridge

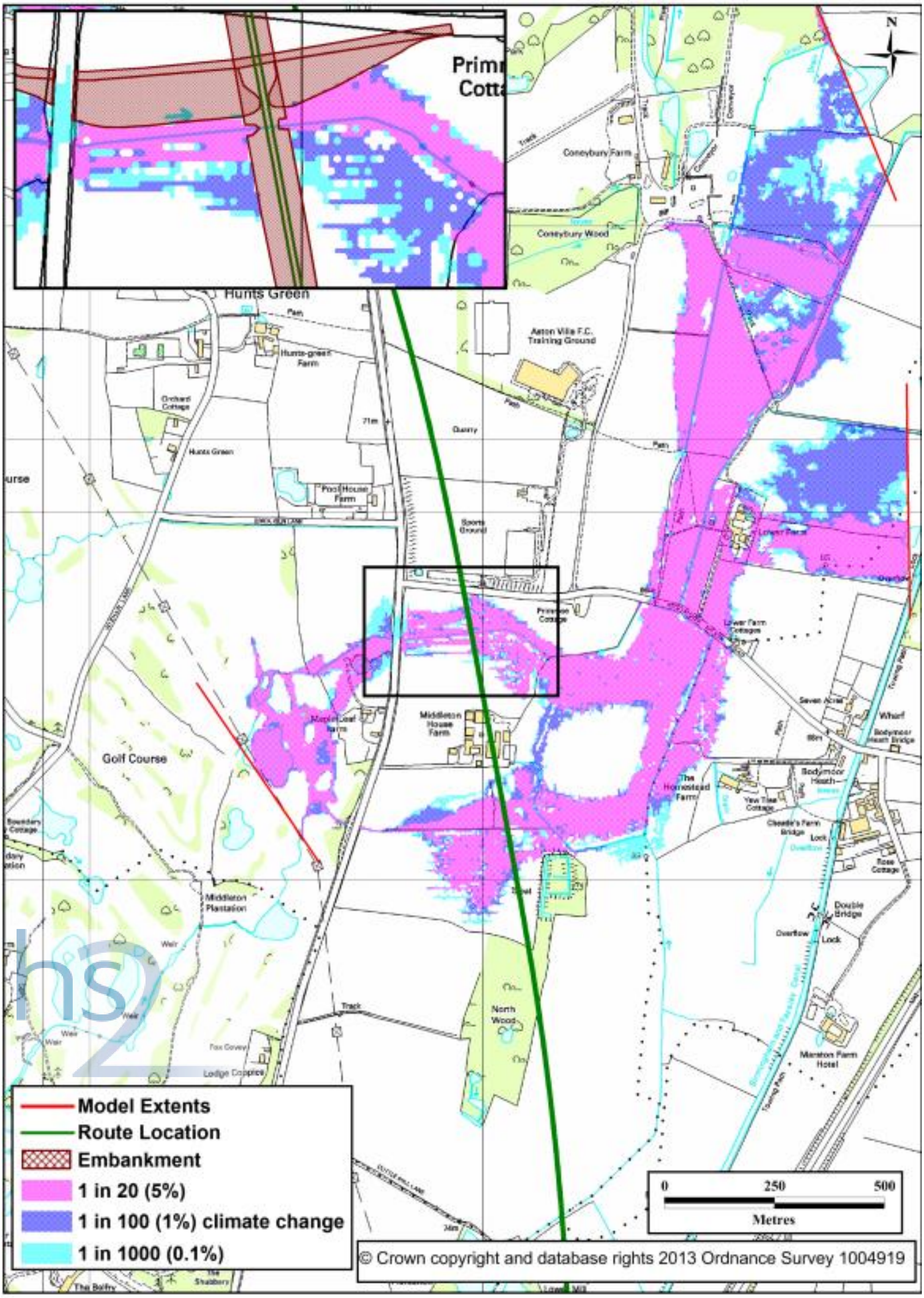
Hydraulics

2.6.3 The Environment Agency confirmed that there were no existing hydraulic models available for this particular watercourse. A TUFLOW model was built on a 5m grid resolution. The two dimensional domain covered the floodplain of three watercourses at the crossings of Cuttle Mill underbridge, North Wood north culvert and Hunts Green underbridge. The reason for their inclusion was the hydraulic interaction of these watercourses downstream of the crossing. The lateral extents of the model were defined by the coverage of available 1m resolution LiDAR data. A Manning's roughness coefficient of 0.04 was adopted for the floodplain and the watercourse. An HQ polyline layer has been used for the downstream boundary, based on the slope of the floodplain at that location; which was a 1 in 200 slope in this case.

2.6.4 The watercourse was not explicitly represented within the model but instead the LiDAR DTM was used to provide the channel geometry. The watercourse depth was defined by the LiDAR DTM and the width limited to the cell size width of 5m. Therefore, the watercourse geometry is not well defined, the consequence of which is an underestimate of the channel conveyance and hence, overestimate the floodplain inundation.

2.6.5 The watercourse reach upstream of the A4091 Tamworth Road Bridge was refined further to improve the flow route. This was achieved by using a GIS polyline layer or Z-line layer to smoothen the channel bed topography. In addition, the attenuation upstream of the A4091 Road Bridge was modelled by a Z-line layer representing the road culvert.

Figure 10: Crossing location and flood extents for Hunts Green underbridge



2.6.6 In the Proposed Scheme model, the underbridge structure was modelled as 4m x 3.35m culvert using a one dimensional network layer. The culvert unit used the LiDAR DTM to extract upstream and downstream inverts. The route embankment was represented as a Z-shape layer with an arbitrarily high elevation. Another Z-shape layer has been added to represent the Bodymoor Heath Lane realignment. The Proposed Scheme model schematic along with the flood extents are provided within the inset of the flood map in Figure 10.

2.6.7 The models were run at a two second timestep for duration of 20 hours.

2.6.8 It has been assumed that the upstream lake at Mill Pool is full at the time of the flood events and so any attenuation from the upstream lake is not included. Apart from the upstream lake, there are no other physical constraints immediately upstream or downstream of the route crossing.

2.6.9 The baseline floodplain width at the crossing is 111m for the 1 in 100 (1%) annual probability with an allowance for climate change event. The cross-section with peak flood levels at the crossing is shown in Figure 11. The modelled peak levels and scheme impacts are summarised in Figure 10. The baseline peak velocity contours and scheme impact on velocities for the 1 in 100 (1%) annual probability with an allowance for climate change event are also provided in Figure 12.

Figure 11: Cross-section with flood levels for Hunts Green underbridge

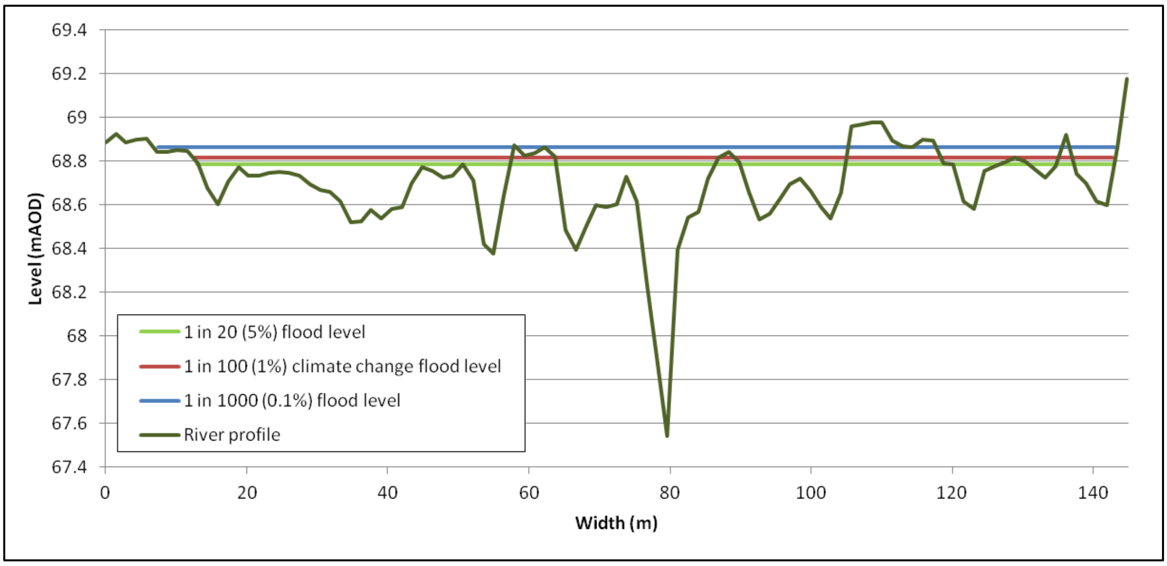
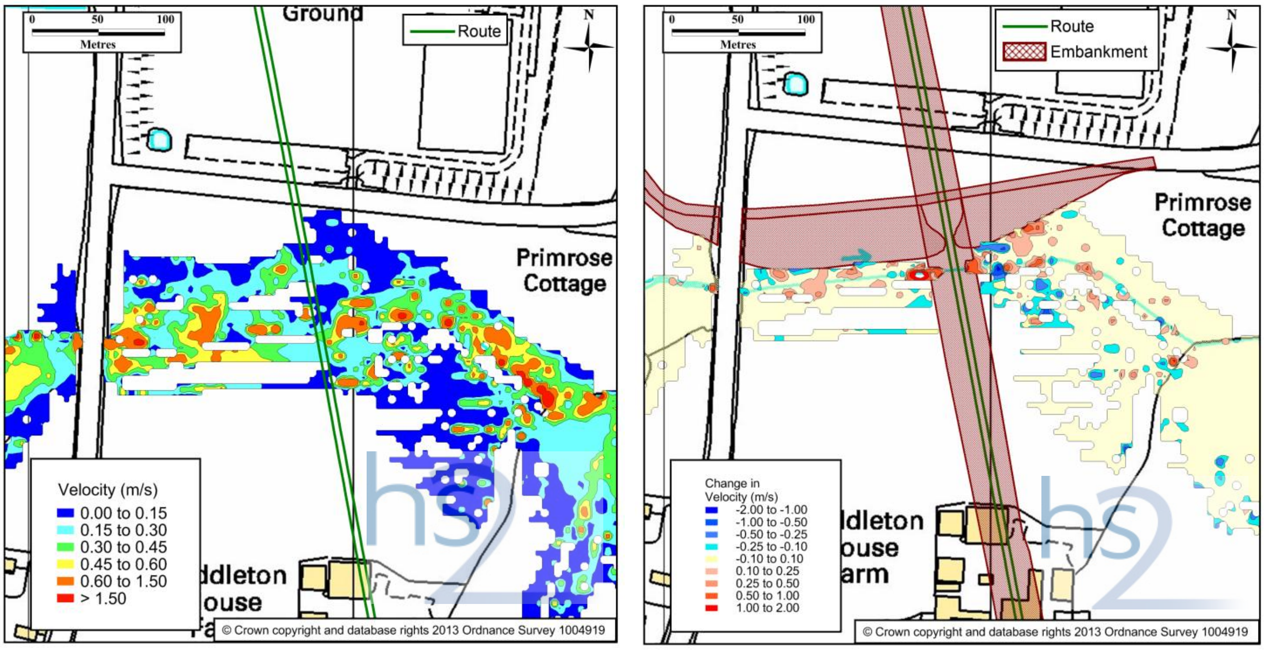


Table 10: Modelled peak levels for Hunts Green underbridge

Flood event	Peak flood level		Change in flood level
	Baseline	Scheme	
1 in 20 (5%)	68.784mAOD	68.844mAOD	60mm
1 in 100 (1%) climate change	68.817mAOD	68.913mAOD	96mm
1 in 1000 (0.1%)	68.861mAOD	69.035mAOD	174mm

Figure 12: Baseline peak velocity contours and scheme impacts for 1 in 100 (1%) climate change event at Hunts Green underbridge



Sensitivity assessment

2.6.10 Sensitivity assessment was carried out by adding 20% to the 1 in 100 (1%) annual probability with an allowance for climate change and 1 in 1000 (0.1%) annual probability events. Models were run for both the baseline and Proposed Scheme scenarios. The increase in flows showed up to a 96mm rise in scheme peak levels for the 1 in 100 (1%) annual probability with an allowance for climate change event. However, the soffit level will be sufficiently above the modelled peak levels with sensitivity allowance, providing the design clearance of 600mm. There is a change in flood extents of up to 19% in the upstream reach between the crossing and the A4091 Tamworth Road Bridge. However no additional receptors have been affected apart from agricultural land. Therefore, the impacts of the scheme on flood risk will still be valid with these sensitivity changes.

Conclusions

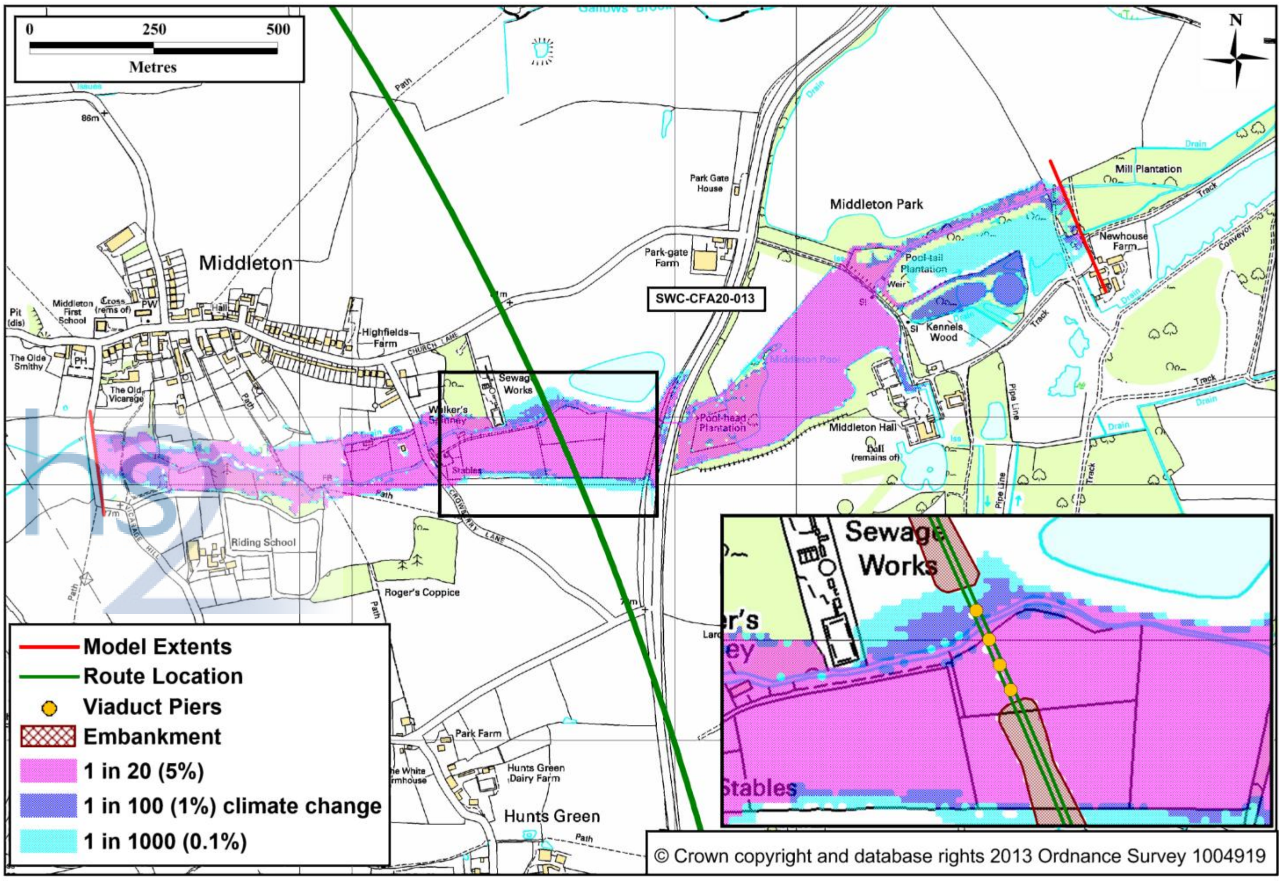
2.6.11 The Proposed Scheme showed up to a 96mm increase in the peak levels for the 1 in 100 (1%) annual probability plus climate change event. The reach showing increases of peak levels of greater than 10mm is limited to 135m which is downstream of the A4091. A replacement flood storage area was identified upstream of the crossing which mitigated the increase in peak levels, as confirmed with modelling. With appropriate reprofiling of the floodplain, the amount of flow into the flood storage area could be controlled to provide effective mitigation. With mitigation, the scheme shows an increase of 32mm for the 1 in 100 (1%) annual probability with an allowance for climate change event, which is a minor impact.

2.6.12 There is a localised increase in velocities of up to 0.55m/s about 90m downstream of the crossing and minimal changes elsewhere. Given the constraints of the A4091 Tamworth Road Bridge upstream, the Bodymoor Heath Lane realignment caused this localised increase in velocities downstream.

2.7 Langley Brook viaduct

2.7.1 The crossing consists of a viaduct structure of about 90m wide crossing Langley Brook, SWC-CFA20-013 (Volume 5: Map WR-05-058, F6).The watercourse flows from west of the crossing and continues east within the model extents as shown in Figure 13.

Figure 13: Crossing location and flood extents for Langley Brook viaduct



Hydrology

2.7.2 The river inflow hydrology was defined using the ReFH method. The hydrological assessment adopted the general methodology as described in Section 1.2.3 and Section 1.2.4 of this report. Details of the ReFH methodology are provided in Section 0 of this report. The estimated peak flows were set as hydrographs as an inflow boundary to the hydraulic model. The flow values are provided in Table 11.

Table 11: Hydrology results: modelled inflows for Langley Brook viaduct

Watercourse identifier	Environment Agency Flood Zone	1 in 20 (5%) flow	1 in 100 (1%) climate change flow	1 in 1000 (0.1%) flow	Modelled structure
SWC-CFA20-013	3	8.26m^3/s	14.24m^3/s	21.62m^3/s	Viaduct

Hydraulics

2.7.3 The Environment Agency confirmed that there were no existing hydraulic models available for this particular watercourse. Therefore, a TUFLOW model was built on a 5m cell resolution. The two dimensional domain covered the floodplain of the

watercourse, the extents of which were defined by the available LiDAR data resized at a 1m resolution. The inflow to the watercourse was applied upstream using a TUFLOW boundary condition polyline layer, linking it to a flow time series within a boundary condition database. A Manning's roughness coefficient of 0.04 was adopted for the floodplain and the watercourse based on an inspection of aerial photography. An HQ polyline layer has been used for the downstream boundary, based on the slope of the floodplain at that location; which was a 1 in 200 slope in this case.

2.7.4 The watercourse was not explicitly represented within the model but instead the LiDAR DTM was used to provide the channel geometry. The watercourse depth was defined by the LiDAR DTM and the width limited to the cell size width of 5m. Therefore, the watercourse geometry is not well defined, the consequence of which is an underestimate of the channel conveyance and hence, an overestimation of the floodplain inundation.

2.7.5 The viaduct structure was modelled by providing a gap in the route embankment which was represented by Z-polygon layers. The soffit levels were not added into the model. This was because the 1 in 1000 (0.1%) annual probability modelled peak flood levels, along with sufficient clearance, would form the basis of designing the soffit heights. Piers were modelled during the design development but they had negligible impact on the peak levels. The Proposed Scheme model schematic along with the flood extents are provided within the inset of the flood map in Figure 13.

2.7.6 The resulting models were run at a two second timestep for a duration of 25 hours to cover the duration of the inflow hydrograph.

2.7.7 There are two key hydraulic constraints to this model. First, the Langley Brook passes under several road bridges along its length. The A4091 Tamworth Road bridge just downstream of the crossing was assumed to have a significant impact on the hydraulics at the crossing; based on the LiDAR DTM and close proximity of the structure to the crossing. The culvert structure at Vicarage Road 1.2km upstream of the crossing was excluded given the closer proximity and dominant impact of the A4091 Tamworth Road culvert. There was no available information on the type and dimensions of the A4091 Tamworth Road culvert structure and hence this was not modelled explicitly. A Z-polyline layer was added to represent the flow route through the A4091 Tamworth Road culvert structure. The inverts were set at the channel bed levels and the structure width set to the width of the channel. Therefore, there is uncertainty on the impact of this structure on flood risk at the crossing.

2.7.8 Secondly, the Middleton Pool downstream of the A4091 Tamworth Road causes floodplain attenuation which was assumed to have an influence on the flood risk at the crossing. However, this impact will not be as significant as that of the A4091 Tamworth Road embankment and culvert structure. Based on the OS mapping and aerial photographs, it was assumed there were two outlet structures under the road embankment just downstream of Middleton Pool. There was no available information on type and dimension of these structures. Therefore, the flow routes through these structures were represented by Z-polylines. The structure inverts and width were assumed to be the same as the channel bed levels and widths defined by the LiDAR DTM. Therefore, there is uncertainty on the impacts of these structures on flood risk at the crossing.

2.7.9 The baseline floodplain width at the crossing is 167m for the 1 in 100 (1%) annual probability with an allowance for climate change. The cross-section of Langley Brook with peak flood levels is shown in Figure 14. The modelled peak levels and scheme impacts are summarised in Table 12. The baseline peak velocity contour and scheme impacts on velocities on 1 in 100 (1%) annual probability with an allowance for climate change event are also provided in Figure 15.

Figure 14: Cross-section with flood levels for Langley Brook

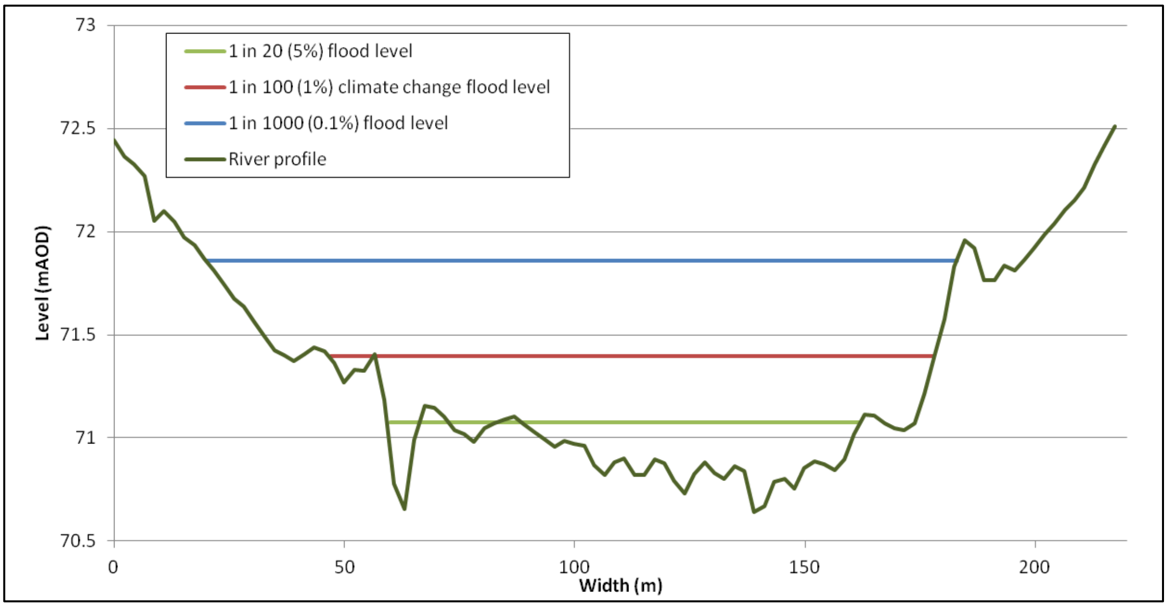
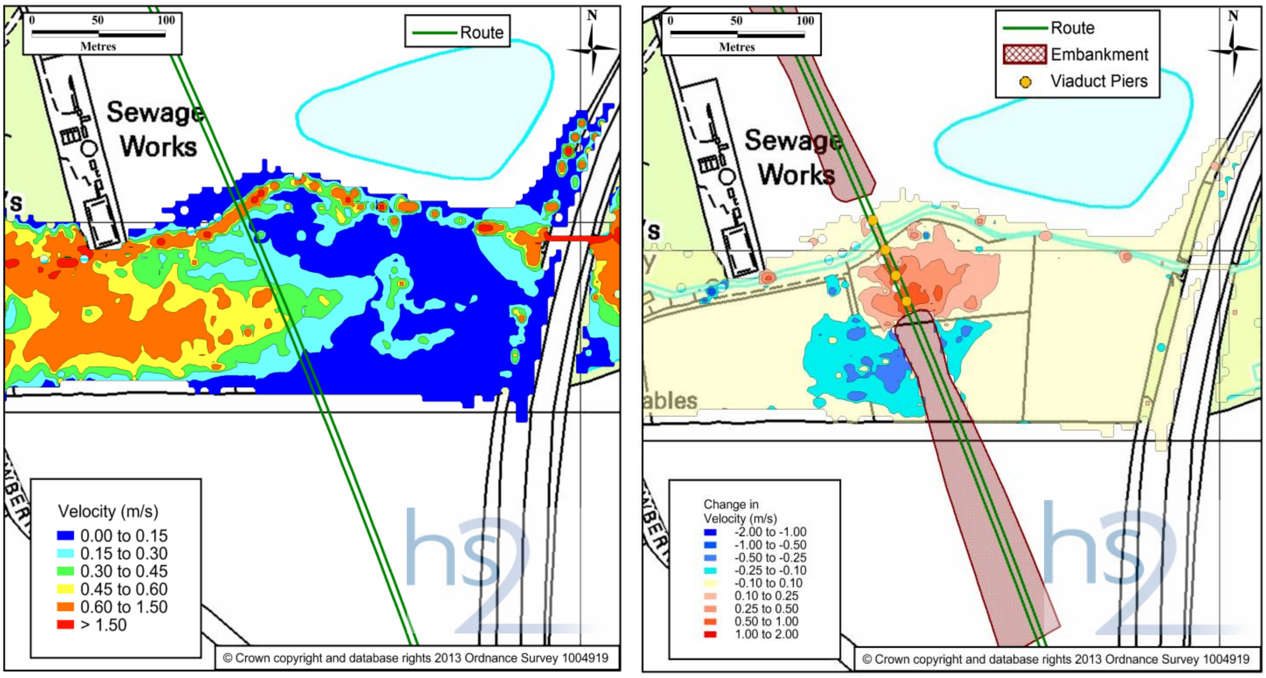


Table 12: Modelled peak levels for Langley Brook viaduct

Flood event	Peak flood level		Change in flood level
	Baseline	Scheme	
1 in 20 (5%)	71.077mAOD	71.157mAOD	80mm
1 in 100 (1%) climate change	71.394mAOD	71.431mAOD	37mm
1 in 1000 (0.1%)	71.859mAOD	71.876mAOD	17mm

Figure 15: Baseline peak velocity contours and scheme impact on velocities for 1 in 100 (%) climate change event at Langley Brook



Sensitivity assessment

2.7.10 Sensitivity assessment was carried out by adding 20% to the 1 in 100 (1%) annual probability with an allowance for climate change and 1 in 1000 (0.1%) annual probability events. Models were run for both the baseline and Proposed Scheme scenarios. The increase in inflows shows up to a 194mm increase in peak levels for the 1 in 100 (1%) annual probability with an allowance for climate change event. However, the soffit level will be sufficiently above the modelled peak levels with sensitivity allowance, providing the design clearance of 600mm.

2.7.11 There is a change in flood extent downstream of Middleton Pool of up to 18% where there are no additional receptors apart from agricultural land. Therefore, the impacts of the scheme on flood risk will still be valid with these sensitivity changes.

Conclusions

2.7.12 The Proposed Scheme increased peak levels by up to 37mm for the 1 in 100 (1%) annual probability with an allowance for climate change event. The increase in peak levels greater than 10mm was limited to a 124m reach upstream of the crossing.

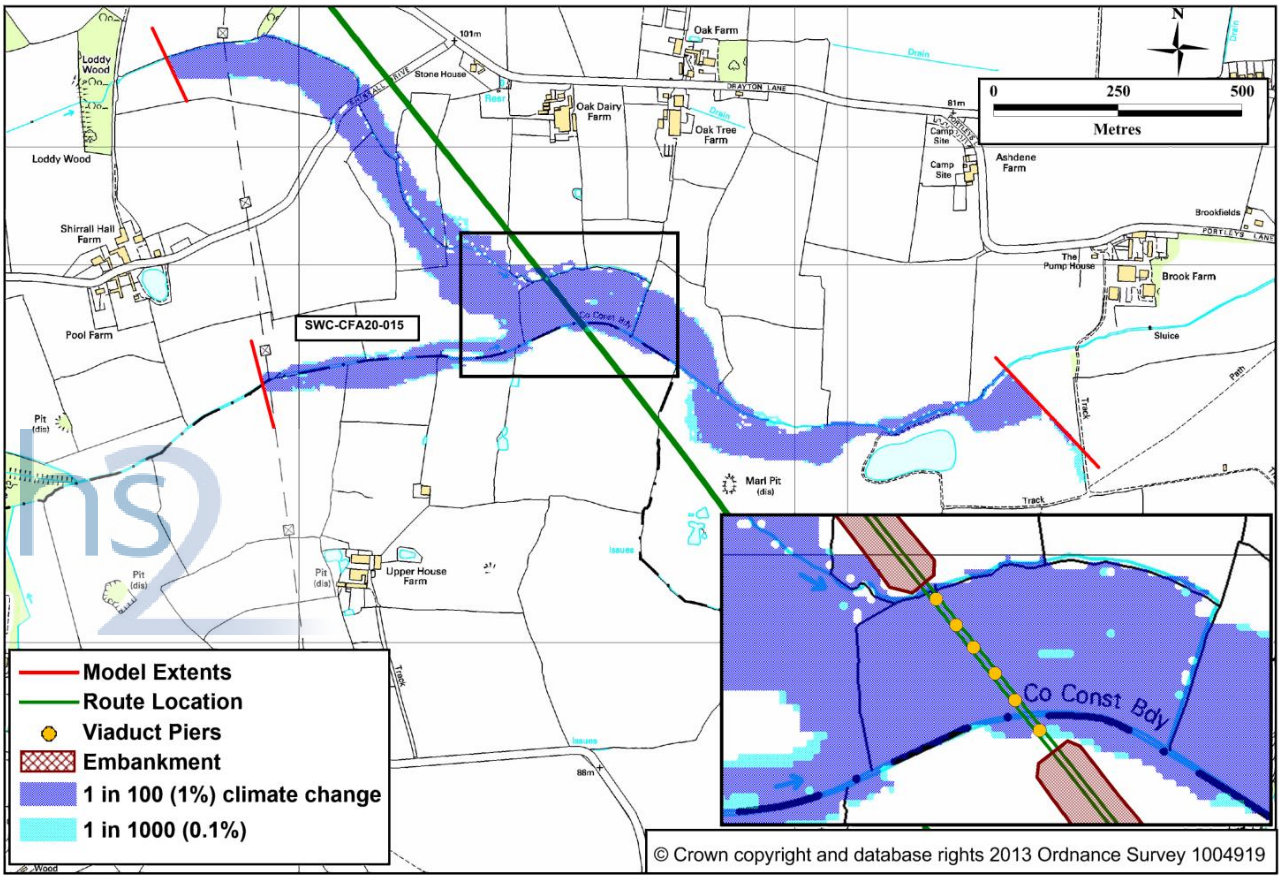
2.7.13 The river modelling has confirmed that a replacement flood storage area upstream of the crossing mitigates the increase in peak levels. With appropriate reprofiling of the floodplain, the amount of flow into the flood storage area could be controlled to provide effective mitigation. With mitigation, the scheme shows a 3mm decrease in peak level for the 1 in 100 (1%) annual probability with an allowance for climate change event.

2.7.14 There are localised increases in velocities of up to 0.66m/s at the structure and minimal changes elsewhere.

2.8 Drayton Bassett viaduct

2.8.1 The crossing consists of a viaduct structure of 155m width crossing the watercourse, SWC-CFA20-015 (Volume 5: Map WR-05-058, B7). Although the flood extents at the crossing are due to the two ordinary watercourses as shown in Figure 16, only the watercourse SWC-CFA20-015 flows within this CFA. The watercourse flows from west of the crossing and continues east as shown in Figure 16.

Figure 16: Crossing location and flood extents for Drayton Bassett viaduct



Hydrology

2.8.2 The river inflow hydrology was defined using the ReFH method. The peak flow from the hydrology calculation was used as a constant inflow into the model. This was to ensure that the defined peak flow across the route was maintained. The nature of the floodplain at this point indicates that flow conveyance is the predominant factor rather than storage, therefore this simplification is reasonable. It should be noted that flows for both watercourses in CFA20 and CFA21 are provided here in Table 13 as they contribute to the flooding at this crossing. The details of the hydrological assessment for watercourse SWC-CFA20-015 are provided in the FEH proforma within Section 0 of this report. Details of the hydrological assessment for SWC-CFA21-003 are provided in the modelling report for CFA21 (Volume 5: Appendix WR-004-014).

Table 13: Hydrology results: modelled flows for the Drayton Bassett viaduct

Watercourse identifier	Environment Agency Flood Zone	1 in 100 (1%) climate change flow (m^3/s)	1 in 1000 (0.1%) flow (m^3/s)	Modelled structure
SWC-CFA20-015	3	2.35	3.66	Viaduct
SWC-CFA21-003	3	3.27	5.08	Viaduct

Hydraulics

2.8.3 The TUFLOW model was built on a 5m cell resolution. The 2D domain covered the floodplain of these watercourses, the extents of which were defined by the available LiDAR data. The inflows to the watercourses were applied upstream using a boundary condition polyline layer, linking it to a constant flow time series within a boundary condition database. The downstream boundary was an HQ polyline layer based on the slope of 0.01 for the floodplain at that location; which was 0.01 in this case. The resulting baseline models were run at a two second timestep over the duration of five hours.

2.8.4 The viaduct structure was modelled by adding FC shape layers which represented the piers. The suggested pier dimension was 2m, however for simplicity a percentage blockage of 100% and a form loss coefficient was added to the cells in the location of the piers. The embankments on either side of the viaduct were modelled as Z-polygon layers. The soffit levels were not added into the model. This was because the 1 in 1000 (0.1%) annual probability modelled peak flood levels, along with sufficient clearance, would form the basis of designing the soffit heights.

2.8.5 Further sensitivity was undertaken on the viaduct piers by increasing the percentage blockage to 50% which would have accounted for temporary works. There was no change in modelled peak levels as a result.

2.8.6 The hydraulic constraints to this model are as follows:

- the OS mapping and aerial photographs indicate that there are no major hydraulic constraints on the watercourses such as culvert or bridge crossings; and
- watercourses SWC-CFA20-015 and SWC-CFA21-003 combine into a single watercourse with a combined floodplain at the crossing. Hence, this assessment of river flood risk includes both the watercourses.

2.8.7 The baseline floodplain width at the crossing is 154m for the 1 in 100 (1%) annual probability with an allowance for climate change. The cross-section of Langley Brook with peak flood levels is shown in Figure 17. The modelled peak levels and scheme impacts are summarised in Table 14. The baseline peak velocity contour and scheme impacts on velocities on 1 in 100 (1%) annual probability with an allowance for climate change event are also provided in Figure 18.

Figure 17: Cross-section with flood levels for Drayton Bassett viaduct

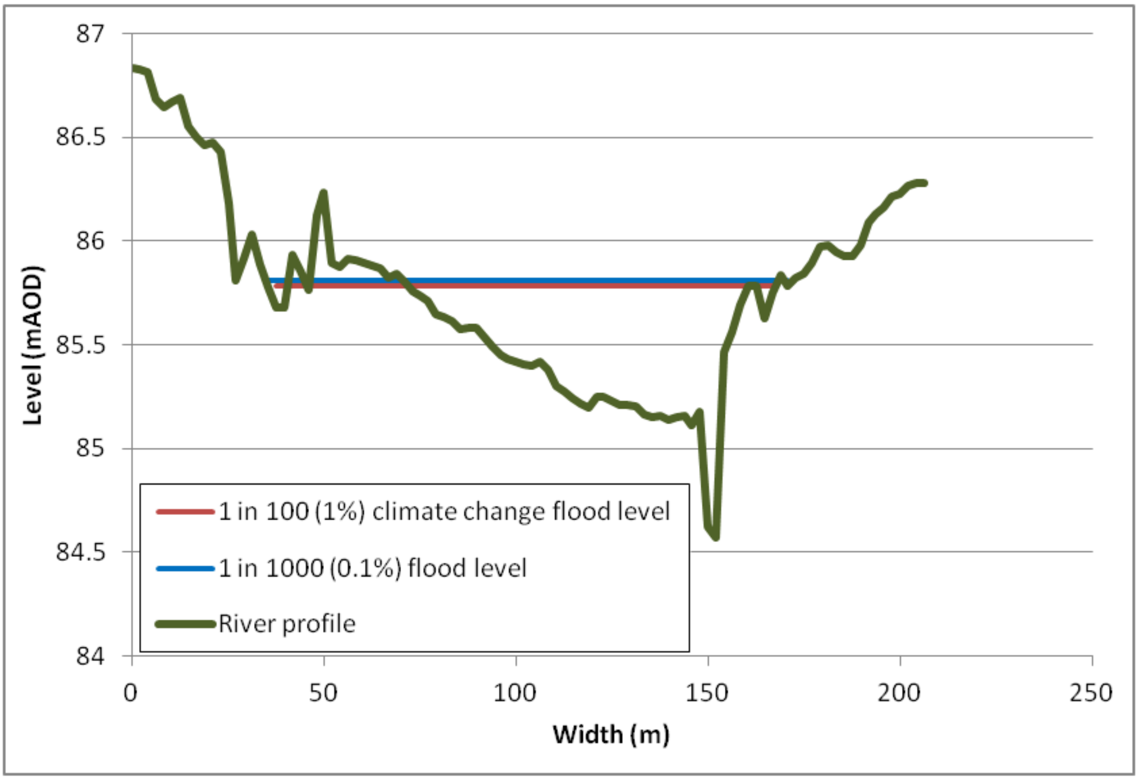
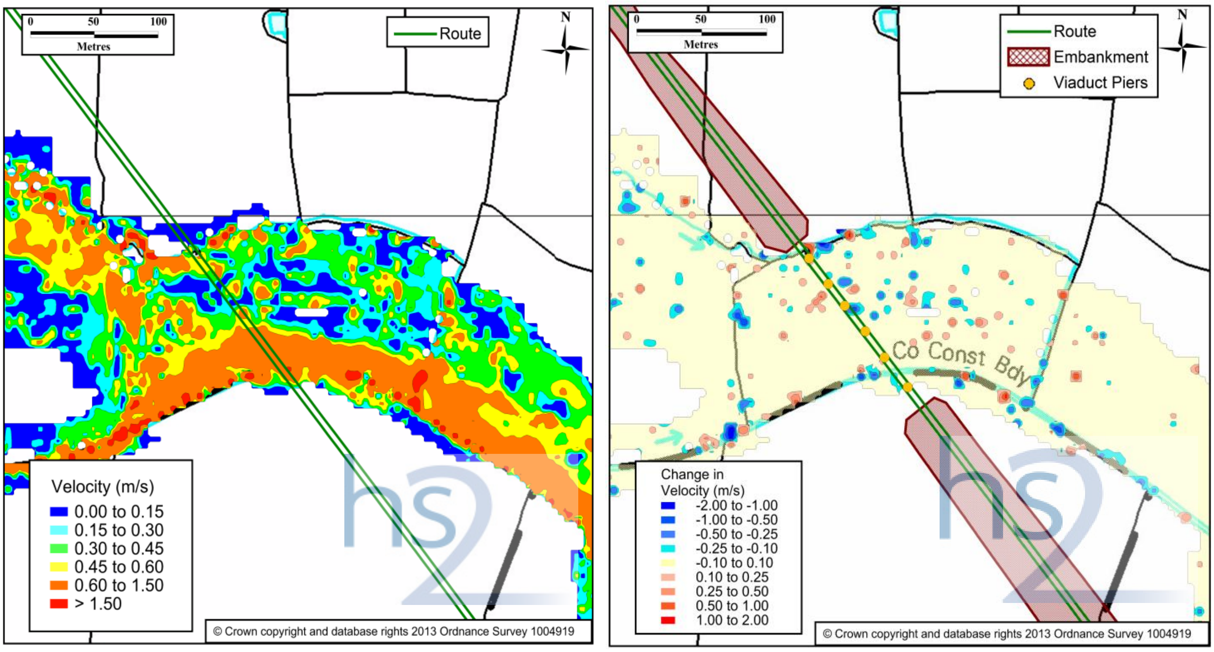


Table 14: Modelled peak levels for the Drayton Bassett viaduct

Flood event	Peak flood level		Δ flood level
	Baseline	Scheme	
1 in 100 (1%) climate change	85.786mAOD	85.786mAOD	0mm
1 in 1000 (0.1%)	85.810mAOD	85.810mAOD	0mm

Figure 18: Baseline peak velocity contours and scheme impact on velocities for 1 in 100 (1%) climate change event at Drayton Bassett viaduct



Sensitivity assessment

2.8.8 Sensitivity assessment was undertaken on the inflows by adding 20% to the design inflows of the 1 in 100 (1%) annual probability with an allowance for climate change and 1 in 1000 (0.1%) annual probability events. Models were run for both the baseline and Proposed Scheme scenarios. For the various scenarios, peak levels increased up to a maximum of 13mm which was considered minimal. Therefore, the impact of the scheme on flood risk will still be valid with these sensitivity changes.

Conclusions

2.8.9 The proposed viaduct structure showed no increase in peak levels for the 1 in 100 (1%) annual probability event. Therefore, the viaduct had no impact on flood risk upstream.

2.8.10 There were no significant increases in velocities due to the viaduct structure.

3 FEH proformas

3.1 Overview

3.1.1 This section provides the FEH Proformas for the hydrological calculations of the various watercourses for which there was no existing hydrology available.

3.1.2 The FEH Proformas are based on the Environment Agency supporting document to the flood estimation guidelines[4].

3.1.3 The watercourse crossings in this study area covered in the FEH Proforma are Curdworth viaduct (SWC-CFA20-002 and SWC-CFA20-003), Cuttle Mill underbridge (SWC-CFA20-009), North Wood north culvert (SWC-CFA20-010), Hunts Green underbridge (SWC-CFA20-011), Langley Brook viaduct (SWC-CFA20-013), Gallows Brook culvert (SWC-CFA20-014) and Drayton Bassett viaduct (SWC-CFA20-015. There are two watercourses at the Drayton Bassett viaduct crossing, however only details of the hydrological calculations for SWC-CFA20-015 have been provided here. Details of the hydrological assessment for SWC-CFA21-003 are provided in the modelling report for CFA21 (Volume 5: Appendix WR-004-014). For Curdworth viaduct, there are two watercourses at the crossing and hydrological assessments were undertaken for each watercourse separately and hence the watercourse identifiers are provided here.

3.1.4 The peak flow for the River Tame for crossing River Tame viaducts was taken from the River Tame Flood Risk Mapping Study8. The FEH proforma of the hydrological assessment is provided within the report for the River Tame Flood Risk Mapping Study8 and is not reproduced within this report.

3.2 Method statement

Overview of requirements for flood estimates

Item	Comments
Give an overview which includes: Purpose of study Approx no. of flood estimates required Peak flows or hydrographs Range of return periods and locations Approx time available	This proforma outlines the hydrological calculations carried out for the assessment of flood risk. As part of the Proposed Scheme, a number of watercourses will require culvert structures under the route and hence it must be ensured that the culvert would be of sufficient capacity. It is vital that the proposed structures are not under designed and hence conservative flows are necessary in line with current requirements of the Proposed Scheme. At a later stage, if a more in-depth assessment determines lower flow, and hence smaller structures would have sufficient capacity, this is acceptable. Flows are required at all watercourse crossings within the study area. The Tame catchment located in this section of the route contains 29 crossings. This particular assessment outlines the derivation of flows at eight of these locations within this study area for the 1 in 20 (5%) annual probability, 1 in 100 (1%) annual probability, 1 in 100 (1%) annual probability with an allowance for climate change and the 1 in 1000 (0.1%) annual probability events.

Overview of catchment

Item	Comments
Brief description of catchment, or reference to section in accompanying report.	The eight catchments assessed here range in size from 0.59km^2 to over 16.46km^2. The catchments range from entirely rural to moderately urbanised and none are classed as highly permeable catchments.

Source of flood peak data

Item	Comments
Was the HiFlows UK dataset used? If so, which version? If not, why not? Record any changes made	No. Only method implemented at this stage is ReFH and hence HiFlows data is not utilised.

Gauging stations (flow or level)

3.2.1 Details of gauging stations at the sites of flood estimates or nearby at potential donor sites.

3.2.2 Local donor sites have been sought however in most cases the catchment area of the subject catchment was found to be significantly smaller than that of any potential local donor.

Watercourse	Station name	Gauging authority number	National River Flood Archive number (used in FEH)	Grid reference	Catchment area (km²)	Type (rated/ultrasonic/ level)	Start and end of flow record
Not applicable							

Data available at each flow gauging station

Station name	Start and end of data in HiFlows UK	Update for this study?	Suitable for QMED	Suitable for pooling?	Data quality check needed?	Other comments on station and flow data quality e.g. information from HiFlows-UK, trends in flood peaks, outliers	
Not applicable							
Give link/reference to any further data quality checks carried out							

Rating equations

Station name	Type of rating e.g. theoretical, empirical; degree of extrapolation	Rating review needed	Reasons – e.g. availability of recent flow gaugings, amount of scatter in the rating.
Not applicable			
Give link/reference to any rating reviews carried out			

Other data available and how it has been obtained

Type of data	Data relevant to this study	Data available?	Source of data and licence reference if from EA	Date obtained	Details
Check flow gaugings (if planned to review ratings)	No				
Historic flood data – give link to historic review if carried out.	No				
Flow data for events	No				
Rainfall data for events	No				
Potential evaporation data	No				
Results from previous studies	No				
Other data or information (e.g. groundwater, tides)	No				

Initial Choice of approach

Is FEH appropriate? (it may not be for very small, heavily urbanised or complex catchments) If not, describe other methods to be used.	ReFH has been utilised for this assessment, as a quick method for determining flows, given the high number of locations requiring flood estimates within the study area. At the beginning of the modelling study this method was specifically aimed at determining at which locations the proposed culvert or bridge would have sufficient capacity for flood flows, and hence whether the Proposed Scheme design requires mitigation. This assessment does not include heavily urbanised catchments, highly permeable catchments or complex ones. There are a number of small catchments within the assessment. The current Environment Agency FEH guidance (2012)[9] suggests that the FEH methods should now be preferred for the estimate of flow in small catchments, in comparison to older methods. In cases where catchments are not represented on the FEH CD-ROM, a scaling method based on area has been used.
Outline the conceptual model, addressing questions such as: **Where are the main sites of interest?** **What is likely to cause flooding at those locations? (peak flows, flood volumes, combinations of peaks, groundwater, snowmelt, tides…)** **Might those locations flood from run-off generated on part of the catchment only, e.g. downstream of a reservoir?** **Is there a need to consider temporary debris dams that could collapse?**	The main sites of interest are at the crossing locations and hence are the points at which flow has been derived. Each point at which flow has been derived has been named in accordance with the associated watercourse identifier at the crossings. At this stage it is considered that peak flows are likely to be the main cause of flooding, following development, due to the potentially constricting culvert or bridge. One crossing, Cuttle Mill underbridge, is located downstream of a reservoir, however the operation of the reservoir is unknown at this stage and hence the presence of the reservoir has been ignored for the purposes of this hydrological assessment. None of the other crossings would be impacted by flow from reservoirs. As part of this assessment it is not deemed necessary to consider the risk of a temporary dam collapse.

[9] Environment Agency (2012), *Flood estimation guidelines* (197_08).

Any unusual catchment features to take into account? **e.g.** **highly permeable – avoid ReFH if BFIHOST>0.65, consider permeable catchment adjustment for statistical method if SPRHOST<20%** **highly urbanised – avoid standard ReFH if URBEXT1990>0.125; consider FEH Statistical or other alternatives; consider method that can account for differing sewer and topographic catchments** **pumped watercourse – consider lowland catchment version of rainfall-run-off method** **major reservoir influence (FARL<0.90) – consider flood routing** **extensive floodplain storage – consider choice of method carefully**	All eight catchments have a FARL >0.9, none are highly permeable and none are classed as highly urbanised.
Initial choice of method(s) and reasons **Will the catchment be split into subcatchments? If so, how?**	ReFH has been used as the only method for determining flows at each location. For the purposes of this assessment it was assumed that the catchment descriptors and boundaries as output from the FEH CD-ROM are accurate and hence no manual adjustment was carried out. Where catchments were not defined in FEH a scaling factor, based on area has been utilised.
Software to be used (with version numbers)	FEH CD-ROM v3.0[10] ISIS Free 3.3

3.2.3 The table below lists the locations of subject sites. The site codes listed below are used in all subsequent tables to save space.

Summary of subject sites

Site code	Watercourse	Site	Easting	Northing	Area on FEH CD-ROM (km^2)	Revised catchment area if altered
SWC-CFA20-002	Ordinary watercourse (tributary of the River Tame)	Route crossing at structure shown in the site code column	419,100	291,790	9.04	Not altered
SWC-CFA20-003	Ordinary watercourse (tributary of the River Tame)	Route crossing at structure shown in the site code column	-	-	-	Catchment not available in FEH CD-ROM and so area calculated manually. Refer to section 'Scaling for catchment not represented on the FEH CD-ROM'.

[10] FEH CD-ROM v3.0 © NERC (CEH). © Crown copyright. © AA. 2009. All rights reserved.

Site code	Watercourse	Site	Easting	Northing	Area on FEH CD-ROM (km²)	Revised catchment area if altered
SWC-CFA20-009	Ordinary watercourse (tributary of Langley Brook)	Route crossing at structure shown in the site code column	419,150	295,200	3.07	Not altered
SWC-CFA20-010	Ordinary watercourse (tributary of Langley Brook)	Route crossing at structure shown in the site code column	419,070	296,120	5.57	Not altered
SWC-CFA20-011	Ordinary watercourse (tributary of Langley Brook)	Route crossing at structure shown in the site code column.	418,970	296,560	2.34	Not altered
SWC-CFA20-013	Ordinary watercourse (Langley Brook)	Route crossing at structure shown in the site code column	418,500	298,140	16.46	Not altered
SWC-CFA20-014	Ordinary watercourse (Gallows Brook)	Route crossing at structure shown in the site code column	417,820	299,260	0.59	Not altered
SWC-CFA20-015	Ordinary watercourse (tributary of the River Tame)	Route crossing at structure shown in the site code column	417,600	299,600	2.08	Not altered
Reasons for choosing above locations		Locations the Proposed Scheme will cross the respective watercourses.				

Important catchment descriptions at each subject site (incorporating any changes made)

Site code	FARL	PROPWET	BFIHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	SPRHOST	URBEXT 2000	FPEXT
SWC-CFA20-002	1.000	0.31	0.551	4.03	21.6	663	36.91	0.063	0.191
SWC-CFA20-009	0.917	0.31	0.630	2.09	16.3	656	39.2	0.004	0.148
SWC-CFA20-010	0.989	0.31	0.421	2.99	28	664	38.14	0.023	0.090
SWC-CFA20-011	1.000	0.31	0.377	1.46	20.5	657	38.37	0.000	0.136
SWC-CFA20-013	0.969	0.31	0.433	4.94	35.8	685	36.15	0.080	0.060
SWC-CFA20-014	1.000	0.31	0.355	0.73	29.1	667	42.85	0.002	0.081
SWC-CFA20-015	1.000	0.31	0.389	2.04	32.2	681	39.40	0.000	0.047

Checking catchment descriptors

Record how catchment boundary was checked and describe any changes (refer to maps if needed)	Catchment boundaries were not checked, it was assumed that catchment boundaries as shown on the FEH CD-ROM were accurate. Where detailed assessment is required at a later stage, it is recommended that catchment boundaries are fully checked. This may result in different flows than those outlined within this proforma. Catchments not represented on the FEH CD-ROM were determined using OS and topographic mapping.
Record how other catchment descriptors (especially soils) were checked and describe any changes. Include before/after table if necessary	No further checking of catchment descriptors was carried out.
Source of URBEXT	URBEXT1990
Method for updating of URBEXT	CPRE formula from FEH Volume 4

3.3 Revitalised flood hydrograph (ReFH) method

Parameters for ReFH model

3.3.1 Note: If parameters are estimated from catchment descriptors, they are easily reproducible so it is not essential to enter them in the table.

Site code	Method OPT: Optimisation BR: Baseflow recession fitting CD: Catchment descriptors DT: Data transfer (give details)	Tp (hours Time to peak)	Cmax (mm) Maximum storage capacity	BL (hours) Baseflow lag	BR Baseflow recharge
SWC-CFA20-002	CD	4.45	489	40	1.292
SWC-CFA20-009	CD	3.93	509	44	1.494
SWC-CFA20-010	CD	3.94	348	37	0.966
SWC-CFA20-011	CD	3.01	313	32	0.858
SWC-CFA20-013	CD	4.15	357	35	0.996
SWC-CFA20-014	CD	1.79	296	27	0.804
SWC-CFA20-015	CD	3.24	322	35	0.887
Brief description of any flood event analysis carried out (further details should be given below or in a project report)			None at this stage of the assessment.		

Note: only the catchments which are represented on the FEH CD-ROM have been included in the table above.

Design events for ReFH method

Site code	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)	Storm area for ARF (if not catchment area)
SWC-CFA20-002	Rural	Winter	7.3	0.960
SWC-CFA20-009	Rural	Winter	6.5	0.971
SWC-CFA20-010	Rural	Winter	6.5	0.964

Site code	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)	Storm area for ARF (if not catchment area)
SWC-CFA20-011	Rural	Winter	4.9	0.971
SWC-CFA20-013	Rural	Winter	6.9	0.950
SWC-CFA20-014	Rural	Winter	2.9	0.978
SWC-CFA20-015	Rural	Winter	5.5	0.973
Are the storm durations likely to be changed in the next stage of the study, e.g. by optimisation within a hydraulic model?			Storm durations will not be altered during the next stage of the hydraulic modelling.	

Flood estimates from the ReFH method

Site code	Flood peak (m^3/s) for the following flood event				Scaling factor
	1 in 20 (5%)	1 in 100 (1%)	1 in 100 (1%) climate change[11]	1 in 1000 (0.1%)	1 in 100 (1%)
SWC-CFA20-002	3.15	4.52	5.42	8.23	0.50
SWC-CFA20-009	0.85	1.25	1.50	2.37	0.41
SWC-CFA20-010	2.87	4.13	4.96	7.54	0.74
SWC-CFA20-011	1.51	2.20	2.64	4.09	0.94
SWC-CFA20-013	8.26	11.87	14.24	21.62	0.72
SWC-CFA20-014	0.54	0.81	0.97	1.54	1.37
SWC-CFA20-015	1.35	1.96	2.35	3.66	0.94

Scaling for catchment not represented on the FEH CD-ROM

Site code	Manual catchment area (km^2)	Scaled flows 1 in 100 (1%) climate change	Scaled flows 1 in 1000 (0.1%)
SWC-CFA20-003	0.21	0.38	0.73
The flows in this table were estimated using the largest scaling factor as determined in the 'Flood estimates from the ReFH method' table and a 10% allowance for data error. This ensures that the values flows estimated for these catchments not represented on the FEH CD-ROM are conservative.			

3.4 Discussion and summary of results

Comparison of results from different methods

3.4.1 This table compares peak flows from various methods with those from the FEH Statistical method at example sites for two key return periods. Blank cells indicate that results for a particular site were not calculated using that method.

[11] The 1 in 100 (1%) annual probability flow with an allowance for climate change is the 1 in 100 (1%) annual probability flow factored by 1.2.

Site Code	Ratio of peak flow to FEH Statistical peak					
	1 in 2 (50%)			1 in 100 (1%)		
	ReFH	Other method	Other method	ReFH	Other method	Other Method
	Not applicable			Not applicable		

Only the ReFH method was carried out as part of this assessment.

Final choice method

Choice of method and reasons – include reference to type of study, nature of catchment and type of data available.	The ReFH method was carried out for the seven catchments represented on the FEH CD-ROM. Flows for the one catchment (SWC-CFA20-003) not represented on the FEH CD-ROM were determined by using a scaling factor based on the estimated flows for the other (FEH CD-ROM represented) catchments. The scaling took the most conservative value and also included an allowance for data error.

Assumptions, limitations and uncertainty

List the main assumptions made (specific to this study)	The FEH CD-ROM accurately represented the catchment boundaries and catchment descriptors. The scaling factor is considered a conservative approach and hence produces conservative flow estimates for the catchment not represented on the FEH CD-ROM.
Discuss any particular limitations, e.g. applying methods outside the range of catchment types or return periods for which they were developed	None
Give what information you can on uncertainty in the results – e.g. confidence limits for the QMED estimates using FEH 3 12.5 or the factorial standard error from Science Report SC050050 (2008).	There is some uncertainty with the results based on the assumptions listed above, however it is considered that the results are conservative and hence would be over-estimating, rather than under-estimating flows.
Comment on the suitability of the results for future studies, e.g. at nearby locations or for different purposes.	The results have been completed for the purposes of the assessment of flood risk as the proposed crossings. The results should not be used for other studies with the exception for comparative purposes.
Give any other comments on the study, for example suggestions for additional work.	When the assessment moves to the detailed design phase it may be useful that the catchment boundaries are checked against LiDAR, OS mapping and other such sources. It is also recommended that the FEH Statistical method is carried out, particularly for high risk crossings. If possible the FEH Statistical method should be carried out for all catchments for comparative purposes and to provide a greater level of confidence with the results.

Checks

Are the results consistent, for example at confluences?	Not applicable – separate catchments assessed.
What do the results imply regarding the return periods of floods during the period of record?	Not applicable
What is the 100-year growth factor? Is this realistic? (The guidance suggests a typical range of 2.1 to 4.0)	Not determined.
If 1000-year flows have been derived, what is the range of ratios for 1000-year flow over 100-year flow?	Varying between 1.82 and 1.90.
What range of specific run-offs (l/s/ha) do the results equate to? Are there any inconsistencies?	Different catchments so not comparable.
How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	None.
Are the results compatible with the longer-term flood history?	None.
Describe any other checks on the results	None.

Final results

Site code	**Flood peak (m^3/s) for the following flood events**			
	1 in 20 (5%)	**1 in 100 (1%)**	**1 in 100 (1%) climate change**	**1 in 1000 (0.1%)**
SWC-CFA20-002	3.15	4.52	5.42	8.23
SWC-CFA20-003	-	-	0.38	0.73
SWC-CFA20-009	0.85	1.25	1.50	2.37
SWC-CFA20-010	2.87	4.13	4.96	7.54
SWC-CFA20-011	1.51	2.20	2.64	4.09
SWC-CFA20-013	8.26	11.87	14.24	21.62
SWC-CFA20-014	0.54	0.81	0.97	1.54
SWC-CFA20-015	1.35	1.96	2.35	3.66
If flood hydrographs are needed for the next stage of the study, where are they provided? (e.g. give filename of spreadsheet, name of ISIS model, or reference to table below)				Hydrographs were provided where necessary.